U. of FLA. LIBRARIES

Second Pacific Area Meeting Papers

Symposium on

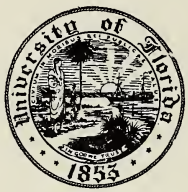
FULL-SCALE TESTS ON HOUSE STRUCTURES

Published by the
AMERICAN SOCIETY FOR TESTING MATERIALS
1916 Race St., Philadelphia 3, Pa.

620.112
A512st
no.210

ASTM Special Technical Publication No. 210

UNIVERSITY
OF FLORIDA
LIBRARIES



SYMPOSIUM ON FULL-SCALE TESTS ON HOUSE STRUCTURES

Presented at the

SECOND PACIFIC AREA NATIONAL MEETING

Los Angeles, California, September 18, 1956

AMERICAN SOCIETY FOR TESTING MATERIALS

57-4789

Reg. U. S. Pat. Off.

ASTM Special Technical Publication No. 210

Price $2.50; to Members $1.85

Published by the
AMERICAN SOCIETY FOR TESTING MATERIALS
1916 Race St., Philadelphia 3, Pa.

FOREWORD

Full scale tests on house structures, though expensive and time consuming, are considered very essential and worthwhile in providing data for economic and adequate design. There are many problems of load application in simulating natural conditions and considerable instrumentation involved in these tests.

Papers and discussions dealing with these problems were presented in a session of the Second Pacific Area National Meeting of the American Society for Testing Materials held in Los Angeles, Calif., September 16–21, 1957.

The symposium was sponsored jointly by Committees D-7 on Wood and E-6 on Methods of Testing Building Constructions. R. F. Legget, National Research Council of Canada, was chairman of the symposium committee and served as chairman of the technical session.

CONTENTS

	Page
Introduction—Robert F. Legget	1
Rigidity and Strength of Houses Built of Plywood Stressed-Cover Panels—R. F. Luxford and E. C. O. Erickson	3
Discussion	12
Navy Facilities for Evaluating Prefabricated Buildings—J. E. Dykins	13
Discussion	19
Full-Scale Testing of Prefabricated Military Buildings—Raymond F. Bartelmes	21
Structural Test of a House Under Simulated Wind and Snow Loads—D. B. Dorey and W. R. Schriever	29
Discussion	48
Full-Scale Tests of Pre-Cast Multi-Story Flat Construction—A. J. Francis, W. P. Brown, and S. Aroni	50
Discussion	60

NOTE.—The Society is not responsible, as a body, for the statements and opinions advanced in this publication.

COPYRIGHT, 1957
BY THE
AMERICAN SOCIETY FOR TESTING MATERIALS

Printed in Baltimore, Md.
July, 1957

SYMPOSIUM ON THE FULL-SCALE TESTS ON HOUSE STRUCTURES

INTRODUCTION

By Robert F. Legget[1]

House design and construction have been receiving increased attention during recent years as the volume of housing in all countries, and particularly in North America, has rapidly increased and as new systems of house construction have been introduced. The growing use of prefabrication as a major factor of the house building industry has served still further to direct the attention of architects and engineers to the development of rational methods for the structural design of house frames. Correspondingly, the desire to reduce the size of members and other details of standard house designs to the minimum consistent with safety and convenience, in the steady search for economy in house construction, has further stimulated research into the strength of house frames.

ASTM Committee E-6 on Methods of Testing Building Construction has a vital responsibility in this connection since its scope comprehends not only the development of test methods for building components, but also test methods for use with complete structures. ASTM Committee D-7 on Wood has a similar interest in this subject when structures built of wood are involved, as is so often the case with North American houses.

[1] Director of Building Research, National Research Council of Canada, Division of Building Research, Ottawa, Canada; Chairman of Symposium Committee.

It seemed appropriate, therefore that, the two committees should jointly sponsor the presentation of a group of papers describing full-scale tests which had been carried out on houses or similar structures, as a contribution to the Second Pacific Area National Meeting of the Society in September, 1956 in Los Angeles, Calif. The importance of wood as a building material on the West Coast gave added emphasis to this subject.

The five papers presented probably constitute the first published collection of papers descriptive of such full-scale structural tests of completed house structures. In a surprising and unpremeditated way, they complement one another to a remarkable degree. Study of the papers will show both advantages and disadvantages of this type of full-scale testing and will suggest further lines calling clearly for intensified research, directed towards further economy in the structural design of such buildings.

The complications of scale effect, especially where wood is involved as a building material, and the complex indeterminate character of the structures described in these papers, combine to show clearly why full-scale tests are a necessity in this branch of investigation. Such tests are costly, in time and money, so that everything possible should be gained from each test that is carried out. The appreciation of Committees E-6 and D-7

is therefore here recorded to the authors of these papers for thus sharing publicly the experience they gained for general benefit and the advance of house design.

Special mention must be made of the concluding paper which the Society has kindly agreed to include in the Symposium even though it was received after the official closing date for the West Coast papers. It was, however, received in time to be summarized at the West Coast meeting. It describes an outstanding full-scale test and has the added interest of being perhaps the first paper from Australia to be published by the Society.

RIGIDITY AND STRENGTH OF HOUSES BUILT OF PLYWOOD STRESSED-COVER PANELS

By R. F. Luxford[1] and E. C. O. Erickson[1]

Some years ago the U. S. Forest Products Laboratory pioneered in the use of stressed-cover panels for walls, roofs, and floors of prefabricated houses. Basically, a stressed-cover panel consists of an inner structural framework with a facing glued to each side to form what is virtually a box girder.

Since this type of construction was radically different from conventional construction, the question was raised as to the ability of a house built entirely of panels to resist high wind forces. To answer this question, a panel house was subjected to simulated wind forces.

Construction of House

Each wall panel was 4 by 8 ft in size and consisted of two $\frac{1}{4}$-in. faces of 3-ply Douglas fir plywood glued to $\frac{3}{4}$ by $1\frac{3}{8}$-in. studs spaced approximately 12 in. apart. The over-all panel thickness was $1\frac{7}{8}$ in. The top, bottom, and vertical edge framing members of the panel framework were set in $\frac{3}{4}$ in. from the edges of the plywood faces to provide for structural joints.

The floor and roof panels consisted of 2 by 6-in. joists spaced 24 in. apart and an upper face of $\frac{5}{8}$-in. plywood of 5 plies and a lower face of $\frac{3}{8}$-in. plywood of 3 plies. The panels were 4 ft wide and varied in length from 8 to 14 ft.

Figure 1 illustrates the assembly method for wall, floor, and roof panels.

Floor panels of this type were tested and found capable of sustaining maximum loads of 300 or more lb per sq. ft over a $13\frac{1}{2}$-ft span. Similarly, wall panels under the pressure of a 60-mph gale would develop a fiber stress in the plywood less than one-third the allowable safe stress for the material.

Vertical mullions built up of three pieces glued together were used to connect the wall panels. Into parallel grooves of the mullions were fitted the plywood edges of adjacent panels. Bottom projecting edges of the panel fitted over a rabbeted sill member for connection between wall and sill, and similarly, the top edges received a $\frac{3}{4}$ by $1\frac{3}{8}$-in. member glued to roof panels along wall and partition lines.

The top and the bottom edges of the exterior wall panels were fastened with five $1\frac{1}{4}$-in. No. 8 screws. No mechanical fastening was used along vertical edges of the wall panels. The interior panels were fastened only with a few brads to the roof or floor.

The outside dimensions of the house were 21 ft $\frac{1}{2}$ in. by 29 ft 4 in. The roof was flat and overhung the wall about 1 ft 4 in. on all four sides.

Method of Test

Since the demonstration house was on a temporary foundation, it could not, because of insufficient anchorage and

[1] Engineer, U. S. Forest Products Laboratory, Forest Service U. S. Dept. of Agriculture, Madison, Wis.

the danger of overturning, be subjected to externally applied forces to simulate wind action. Instead, a system of internal forces which could be translated into wind reaction at the roof and floor ends of the wall panels was used.

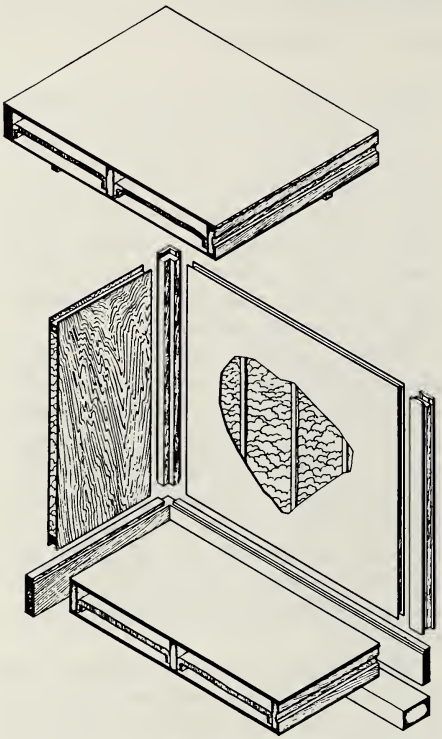


FIG. 1.—The Assembly Method for Wall, Floor, and Roof Panels of Plywood Stressed-Cover Houses.

Testing Apparatus:

Figures 2 and 3 show the test setup, as well as the types of panels in the end walls. Preparatory to test, $\frac{3}{4}$ by $7\frac{1}{2}$-in. filler pieces were tacked to the outside walls between the mullions along the 29-ft sides of the house at the roof line on one side and at the floor line on the opposite side of the house. Two timbers, each 15 ft long, were placed against the filler pieces at each of these positions. Those at the roof line were suspended by two bolts each through the roof panels, while the two at the water table were held in place by metal straps fastened to the building. The timbers projected 1 ft beyond the ends of the building at each corner.

Four $\frac{3}{4}$-in. threaded-rod assemblies were next installed diagonally across the 21-ft width of the building. The rods passed freely through holes bored in the wall 4 in. above the floor and 4 in. below the ceiling and through the timbers. The rods were all in the same plane. Two were located 6 in. beyond the end walls outside the house and two were placed inside the house, spaced 9 ft 6 in. from the outside pair. Each diagonal rod assembly was equipped with one 5000-lb dynamometer, which was hooked into the lower one-third length of the rods. A ball bearing washer unit was supplied between the nut of each rod and the low end timber to permit the easy turning of nuts on the rod threads while producing the diagonal tension forces.

The timbers, which were originally 7 in. square, were beveled on one face at the proper angle for seating the bearing washers.

The filler pieces between the timbers and the wall panel faces caused the force to be applied directly to the wall panels rather than to the vertical mullions.

A telescoping wooden rod was fastened to the corner mullions at each end of the building in approximately the same plane as that of the rods. A scale for obtaining readings of the diagonal shortening of the wall was attached to the wooden rod.

The vertical movement between the end wall panels and their connecting mullions was obtained by measuring the distance between a horizontal line drawn on a gummed label placed at a vertical edge of each wall panel and a nail driven into the mullion. Partition panels parallel

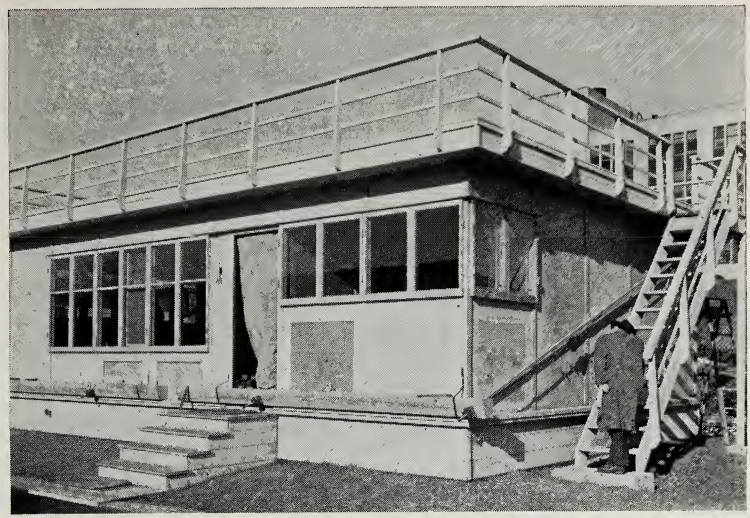


FIG. 2.—Apparatus for Testing Rigidity of House Shown in Place.

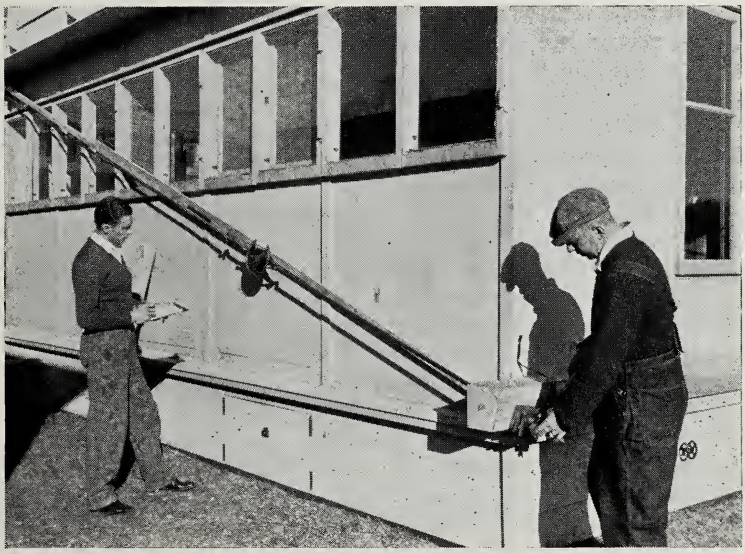


FIG. 3.—Apparatus for Testing Rigidity of Wall Construction. Load Was Applied by Turning Nut on Diagonal Rod. Load Was Measured with Dynamometer, and Diagonal Shortening Was Measured with a Scale on a Telescoping Wooden Bar Placed Parallel to the Rod.

to the end walls were likewise marked and vertical movements observed.

The timbers were of sufficient size and stiffness to prevent any appreciable amount of bending action along the unsupported walls, as well as being capable

loads fell off an average of 15 per cent at each dynamometer. On resumption of the test, the loads were again brought up to 1400 lb at each dynamometer, and all readings were taken and recorded. The loads were then brought up to 3800 lb at

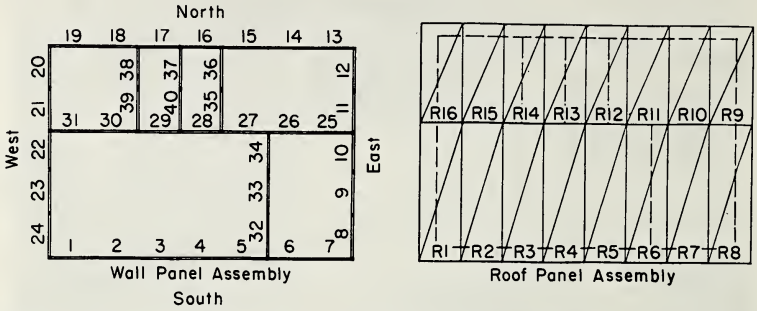


FIG. 4.—Position of Various Panels in the Wall and Roof Assemblies of Plywood Stressed-Cover Test House.

of transmitting load to the partition walls. The north portion of the house had three partition walls of two panels each, and the south portion had one partition wall of three panels (Fig. 4).

Application of Load:

A 200-lb load was first applied to each of the four dynamometers. This load was just sufficient to support and take the principal sag out of the rod assemblies. Load increments of 150 lb were then added to each dynamometer until each read 1100 lb. The diagonal shortening of the end walls and the vertical movement between walls and mullions was recorded for each increment of load. Following the 1100-lb increment, the loads were released until each dynamometer read 200 lb, and all movements were recorded. The load was again returned to 1100 lb at each dynamometer, and readings were taken. From this stage, the load increments were 300 lb.

Following the 1400-lb load readings, the settings of the nuts were left undisturbed for 1 hr. During this interval, the

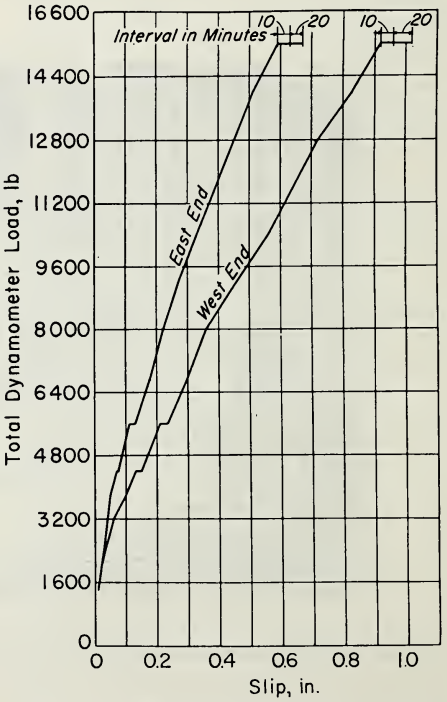


FIG. 5.—Relationship between the Shortening (Slip) of the Two End Diagonals and the Corresponding Loads.

TABLE I.—MOVEMENT OF ASSEMBLED PREFABRICATED PLYWOOD WALL PANELS UNDER RACKING LOAD.

Diagonal Force or Total Load Applied at Dynamometers, lb	Horizontal Wind Load Equivalent,[a] lb per sq ft	Diagonal Shortening of End Walls, in.		Horizontal Movement of Roof, in.	Net Vertical Movement Between Adjacent Panels or at Sides of Panels Adjoining Cross Walls, in.																			
					East-End Wall Panels						West-End Wall Panels						Partition Wall Panels							
		East	West	Southwest corner	12-N	12-11	11-10	10-9	9-8	8-S	20-N	20-21	21-22	22-23	23-24	24-S	36-N	36-35	35-S	34-N	34-33	33-32	32-S
800	4	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1 400	7	0.01	0.01		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2 000	10	0.02	0.02		0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
2 600	13	0.03	0.04		0.00	0.01	0.00	0.00	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.01	0.00	0.02	0.00	0.00	0.02	0.01	0.01
3 200	16	0.04	0.06		0.00	0.01	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.03	0.00	0.00	0.03	0.02	0.01
3 800	19	0.05	0.10		0.00	0.02	0.02	0.00	0.01	0.01	0.01	0.00	0.00	0.01	0.00	0.01	0.01	0.05	0.00	0.00	0.04	0.03	0.01
4 400	22	0.07	0.13		0.00	0.02	0.03	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.00	0.01	0.01	0.06	0.00	0.00	0.05	0.04	0.01
800[b]	4	0.01	0.06		0.00	0.02	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.00	0.00	0.03	0.00	0.01
4 400	22	0.075	0.15		0.00	0.02	0.03	0.01	0.00	0.00	0.01	0.00	0.01	0.01	0.00	0.01	0.02	0.06	0.01	0.00	0.05	0.04	0.01
5 600	28	0.11	0.21		0.01	0.04	0.04	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.02	0.10	0.01	0.00	0.07	0.06	0.02
5 600[c]	28	0.13	0.235		0.01	0.04	0.05	0.04	0.02	0.00	0.01	0.02	0.01	0.02	0.01	0.02	0.03	0.11	0.01	0.00	0.09	0.06	0.02
6 800	34	0.18	0.30	0.00[g]	0.01	0.05	0.07	0.06	0.00	0.01	0.02	0.03	0.02	0.03	0.03	0.02	0.04	0.14	0.02	0.00	0.10	0.09	0.02
8 000	40	0.22	0.36	0.08	0.02	0.07	0.09	0.07	0.02	0.01	0.02	0.03	0.03	0.03	0.03	0.02	0.04	0.17	0.03	0.00	0.13	0.11	0.02
9 200	46	0.27	0.46	0.13	0.03	0.10	0.12	0.10	0.03	0.01	0.02	0.05	0.03	0.05	0.03	0.02	0.07	0.24	0.03	0.01	0.16	0.14	0.03
10 400	52	0.33	0.56	0.17	0.03	0.12	0.15	0.12	0.05	0.02	0.03	0.06	0.05	0.06	0.05	0.03	0.08	0.27	0.04	0.02	0.19	0.17	0.04
11 600	58	0.39	0.64	0.44	0.03	0.13	0.18	0.14	0.06	0.02	0.03	0.07	0.06	0.07	0.06	0.04	0.08	0.33	0.04	0.02	0.25	0.19	0.05
12 800	64	0.45	0.72	0.53	0.03	0.18	0.22	0.17	0.08	0.03	0.02	0.07	0.07	0.08	0.08	0.04	0.10	0.39	0.05	0.03	0.30	0.22	0.05
14 000	70	0.51	0.83	0.63	0.04	0.20	0.27	0.19	0.10	0.03	0.03	0.09	0.08	0.08	0.10	0.04	0.11	0.43	0.06	0.04	0.34	0.26	0.06
15 200	76	0.59	0.92	0.75	0.05	0.24	0.31	0.21	0.11	0.03	0.02	0.10	0.10	0.12	0.10	0.05	0.12	0.50	0.07	0.06	0.38	0.28	0.07
15 200	76	0.63	0.96																				
15 200[d]	76	0.67	1.02	0.75	0.05	0.28	0.35	0.24	0.14	0.04	0.04	0.13	0.14	0.13	0.12	0.05	0.14	0.54	0.07	0.06	0.42	0.32	0.08
0[e]	0	0.24	0.37	0.14	0.03	0.13	0.16	0.12	0.11	0.01	0.02	0.07	0.09	0.08	0.09	0.02	0.08	0.15	0.02	0.01	0.12	0.17	0.04
0[f]	0				0.02	0.11	0.11	0.10	0.09	0.01	0.01	0.05	0.07	0.05	0.05	0.00	0.07	0.11	0.02	0.00	0.09	0.16	0.03

[a] Equals diagonal force divided by 200.0. See appendix for determination of divisor.
[b] The applied force was slowly released to the initial 800 lb starting load, and the corresponding set recorded where measurable.
[c] Measurements recorded after an undisturbed period of 1 hr and restoration of the 15 per cent loss to the previous load.
[d] Measurements recorded after an undisturbed period of 20 min and restoration of the 8 per cent loss to the previous load.
[e] Measurement immediately after the complete release of applied force.
[f] Measurement 18 hr after the applied force had been removed.
[g] The observation of the horizontal roof movement was started at load of 6800 lb.

each dynamometer in increments of 300 lb. During the 10 min required to take the various readings, the loads dropped off considerably, with the drop being greater at the higher loads. At 3800 lb, the loads fell off about 8 per cent. They were brought back to 3800 lb, the diagonal readings were recorded, and the nuts left undisturbed for a period of 20 min. During this time, the loads fell off 2 per cent. Again they were brought up to 3800 lb at each dynamometer, and all readings were taken. The loads were then released to zero, and the readings of set were recorded. Following an overnight period without load, readings were again taken. The set between panels immediately after release of the load was approximately one-half of the total movement. After an overnight period, the set was reduced to approximately one-third of the total movement.

The testing was stopped at the 15,200-lb total load (3800 lb at each dynamometer). At this load, the hooks on the $\frac{3}{4}$-in. mild steel rods began to yield, as indicated by the scaling at the bends.

MOVEMENTS OF PARTS

Distortion of End Walls:

The relationship between shortening of the two end diagonals and the corresponding loads is shown in Fig. 5. Table I gives diagonal shortening values and net vertical movements between adjacent panels for various loads. The panel numbers referred to are shown in Fig. 4.

The three horizontal portions of the curves in Fig. 5 are explained as follows: At 4400 lb the load was slowly released to the 800-lb initial load, and when it was brought back to the former value, an additional movement occurred. At 5600 lb, there was an intermission of 1 hr during which the load dropped and movement increased. When the load was brought back to 5600 lb, further movement resulted. At a maximum load of 15,200 lb, the shorter horizontal portion of the curve represents the movement that occurred during the 10-min recording interval and subsequent restoration of the load. The longer portion represents the further movement that occurred during an undisturbed period of 20 min and the second restoration of the maximum load.

The slip or movement resulting from the restoration of the load after the 20-min period was considerably less than twice as much as occurred after restoring the maximum load following the 10-min period. This behavior indicates that the yielding of the building under the maximum load imposed would ultimately stop.

The west end of the building consisted of five window panels, the upper and lower portions of which were separate spandrel units. These panels offered much less rigidity to the wall than plain panels. This was evidenced by the lesser diagonal shortening of the east end wall, which consisted of three plain panels and two window panels.

Horizontal Movement:

The observation and recording of the horizontal roof movement was not begun until the total load was 6800 lb. The movement was obtained by fastening a scale to the southwest corner of the most westerly roof panel and reading the change from a known starting point on the scale with a transit. The observed movement was 0.75 in. at the maximum test load of 15,200 lb. This was about 0.17 in. more than the value calculated from the corresponding shortening of the diagonal. Just how much horizontal movement occurred during that part of the test before observation was begun is not known. After test, the walls were out of plumb less than $\frac{3}{16}$ in.

Vertical Movement Between Panels:

The racking force produced an independent rotation tendency in each of the end wall panels and the partition panels parallel to them, causing a vertical movement or slip between adjacent panels. While neither the end wall nor partition panels were mechanically fastened to their connecting mullions, the mastic used along the sides and bottom of the beted sill and fastened with five $1\frac{1}{4}$-in. No. 8 screws. This sill had previously been screwed to the floor panels and anchored to the foundation sill. These panels were also screwed to framing members glued to the underside of the roof panels. The screws were symmetrically placed, the center one being at the vertical centerline of the panel. The only other mechanical fastenings were the $1\frac{1}{4}$-in. brads holding a $\frac{1}{2}$-in. square molding

FIG. 6.—Separation of Partition and Ceiling Panels Resulting from Rotation Tendency of Partition Panel Caused by Racking Force.

exterior wall panels, together with friction between the parts, exerted considerable resistance to this movement.

Further resistance to panel rotation would result from the counter-balancing effect of the dead load of the roof and wall panels. In the case of roof weight, only the full dead load of roof panels directly over the end and transverse partition walls were considered active in resisting the racking force.

The bottom projecting edges of exterior wall panels were fitted over a rabstrip in the exterior and interior angles between wall and roof. Some of these brads penetrated the plywood of the wall panels and extended $\frac{1}{2}$ in. into the connector piece glued to the roof panel; others were not driven in such a way as to form a connection between walls and roof. Except for such brads, the partition wall panels were not otherwise fastened to roof or floor.

The rotation tendency was easily seen on those partition wall panels without top molding strips or from which the

strips had been removed prior to the test. The displacement amounted to $\frac{3}{4}$ in. at panel No. 38 (Fig. 6).

Discussion of Results

The diagonal deformation resulting from an applied force equal to 20 lb per sq ft, which is in excess of that often required for assumed wind pressure, was only approximately 0.05 in. on the east end and 0.10 in. on the west end. At the equivalent of 40 lb per sq ft, the corresponding deformations were 0.22 and 0.36 in., and at the maximum load applied, which was equivalent to 76.0 lb per sq ft wind load (Table I), the corresponding deformations were 0.59 and 0.92 in. After the entire load was removed, the permanent set was 0.24 in. for the east end and 0.37 in. for the west end.

While the east wall had less diagonal shortening, there was a greater vertical displacement than at the west wall. The three plain panels and two window panels in the east wall were more rigid than the five window panels in the west wall and permitted less horizontal movement and hence less diagonal shortening.

Except for those panels with windows, distortion was primarily one of rotation of the panels. As would be expected from the small diagonal distortions, the vertical movement between panels was also small. Even at maximum load, the movement was not large, and the displacement between a panel and the adjacent mullion that remained after the load was released averaged 0.04 in. for the west wall, 0.06 in. for the east wall, and 0.06 in. for the partition walls. The displacement between adjacent panels would be twice this amount.

The total computed resisting moment from mechanical fastenings and dead load accounted for about one-half of the moment necessary to counterbalance the net racking moment of 1,023,000 in-lb produced by the maximum force applied (see Appendix for method of calculations).

The results indicate that the resisting factors produced by the manner of assembly provided the other one-half of the necessary resistance to the racking force. These factors included friction and the mastic that was placed in the grooves of the outer-wall mullions, which held the vertical edges of the adjacent wall panels.

A wind pressure of 20 lb per sq ft on the wall and roof area would produce a racking moment of 270,400 in-lb. This moment would be resisted by 189,700 in-lb of moment from dead load of the wall panels in the east and west walls, the partition panels parallel to them, and the roof panels directly over these wall panels. Consequently, only about 80,700 in-lb of resisting moment would be required from mechanical fastenings and friction. According to the computations, the screws alone were capable of developing considerably more than this 80,700 in-lb of resisting moment at 0.01-in. slip, which is within the proportional limit of the screws. It appears evident that the number of screws used was ample, and would have been so with the omission of the mastic between the exterior panels. Additional rigidity can readily be secured, if desired, by using a few nails or screws along the sides of the panels.

Conclusions

1. As a whole, the results indicate a remarkable stability for this building, considering the few fastenings used.

2. The side of the building with the greater number of window openings showed the greater distortion.

3. The resisting moment safely developed by the relatively few screws used in the erection of this building, together with the moment from the dead loads, was ample to resist a wind pressure of 20 lb per sq ft. This was true even when the

resistance developed by the friction between parts or by the mastic placed in the grooves of the outer wall was not considered. Should additional rigidity be required, it can easily be obtained by the use of a few more nails or screws.

4. The permanent set from a diagonal distortion produced by a load approximately four times a 20-lb wind pressure is only about $\frac{1}{3}$ in. Careful visual observations failed to reveal the building as noticeably out of plum in this condition.

APPENDIX

The equivalent wind pressure, racking moment, and resisting moments for dead loads were calculated in the following way:

EQUIVALENT WIND PRESSURE

The width of the end wall measured 21 ft $\frac{1}{2}$ in.; the vertical distance between the two ends of the diagonally placed rods for applying the load measured 7 ft 4 in.; and the length of the rod from wall to wall was 22 ft. $3\frac{1}{2}$ in.

Racking Moment:

Where F equals the diagonal force in pounds, the horizontal component equals

$$\frac{21.04F}{22.29} = 0.944F$$

and the vertical component equals

$$\frac{7.33F}{22.29} = 0.329F$$

Then, the racking moment from horizontal component of applied force equals

$$0.944F \times 88 = 83.1F \text{ in-lb}$$

The resisting moment from vertical compound of applied force equals

$$0.329F \times 48 = 15.8F \text{ in-lb}$$

The 48-in. arm is the width of one panel. The net racking moment equals

$$(83.1 - 15.8)F = 67.3F \text{ in-lb}$$

Using the maximum diagonal force in pounds (F) of 15,200 lb (Table I), the maximum racking moment is

$$67.3 \times 15{,}200 = 1{,}023{,}000 \text{ in-lb}$$

Moment from Wind Pressure:

P equals wind pressure in pounds per square foot. Against the wall panels on the north side (Fig. 4);

$$P \times 29.33 \times 8 \times 48 = P(11{,}260) \text{ in-lb}$$

Against the edge surface of the roof panels on the north side,

$$P \times \left((31.83 \times \frac{8.875}{12} \right) (96) = P(2260) \text{ in-lb}$$

The total moment from wind pressure is $P(13{,}520)$ in-lb.

Equating the moments,

$$67.3F = 13{,}520P$$

$$\frac{F}{P} = \frac{13{,}520}{67.3} = 201$$

$$P = \frac{F}{200} \text{ (approximately)}.$$

Thus, the maximum equivalent wind pressure developed was

$$\frac{15{,}200}{200} = 76 \text{ lb per sq ft}.$$

RESISTING MOMENTS FROM DEAD LOADS

Roof Panels:

The dead load of the roof panels directly over the end or partition wall panels was considered to be active in resisting the applied force. Three 4 by 14-ft panels and five 4 by 10-ft panels, or a total of 368 sq ft, were considered effective. The approximate weight of a 4 by 12-ft panel is 230 lb, or 4.8 lb per sq ft.

Dead load of roof panels is	$4.8 \times 368 =$	1765
Dead load of roof covering is	$4.0 \times 368 =$	1475
The total roof dead load is		3240 lb

With roof dead load acting on a 4-ft moment arm, the resisting moment is

$$3240 \times 48 = 155{,}500 \text{ in-lb}$$

Wall Panels:

The 19 panels in the end walls and partitions parallel to them weighed 75 lb each. Acting on a 2-ft arm, the moment is

$$19 \times 75 \times 24 = 34{,}200 \text{ in-lb}$$

The total resisting moment from the dead loads of the roof and wall panels is

$$155{,}500 + 34{,}200 = 189{,}700 \text{ in-lb}$$

DISCUSSION

MR. PAUL REPLINGER.[1]—Do the authors have any data showing how those deflections and the permanent set would compare with conventional 2 by 4-in. studs with dapped or cut-in braces?

MR. R. F. LUXFORD (*author*).—We have made no similar tests on conventional construction. I know that the question is often asked, "Is a prefabricated house as good as a house with conventional construction, using 2 by 4 studs 16 in. on center?" We feel that these tests indicate that a house of this type of construction ($\frac{1}{4}$-in. plywood) would be very ample for any normal force that would ordinarily be applied to the house, but I cannot answer your question directly.

MR. E. GEORGE STERN.[2]—If a panel would have been omitted, as for a door, could you tell us how it would have influenced the rigidity of the end panels?

MR. LUXFORD.—Actually, the whole panel is not omitted; there is a spandrel panel above the door opening. Ordinarily in calculating the resistance of a wall to racking, door panels are not considered as adding to the over-all resistance. On the other hand, picture windows add somewhat to the rigidity of a wall, as the glass is set tightly into the window opening.

While a door panel is a source of weakness, the floor system and roof system, acting with the walls, apparently give adequate rigidity.

[1] Materials Engineer, California Div. of Highways, San Diego, Calif.

[2] Virginia Polytechnic Inst., Wood Construction, Blacksburg, Va.

NAVY FACILITIES FOR EVALUATING PREFRABRICATED BUILDINGS

By J. E. Dykins[1]

The building test program was inaugurated by the Bureau of Yards and Docks in response to requests by the military services for better prefabricated buildings. The need for such a program became evident during World War II when building demand far outstripped production. To meet this demand, production was stepped up to the point where little or no heed was being given to some of the basic design requirements. The resulting variety of buildings in production by the end of the war clearly pointed to the need for some sort of testing and standardization program.

At the end of the war, the Bureau of Yards and Docks invited interested manufacturers to submit designs for 20 by 48 ft prefabricated buildings. A substantial number of designs were submitted. From these a large number were selected for prototype procurement and evaluation. The U. S. Naval Civil Engineering Research and Evaluation Laboratory at Port Hueneme, Calif. was selected as the test facility.

To establish a test program it was necessary for the laboratory to adopt test criteria and procedure. The criteria and procedure which were adopted during the beginning of the program have since been followed except for minor changes. These included a reduction in some of the loadings, such as reduction of the snow load by 5 lb per sq ft and the floor load by 10 lb per sq ft. The structural loading criteria in present use are (1) snow load, 15 lb per sq ft; (2) floor load, 30 lb per sq ft; and (3) wind load, 70 mph.

Wind load distribution is calculated in accordance with Bureau of Yards and Docks design data plates for wind and earthquake. Figure 1 is a plot of wind velocity in miles per hour *versus* velocity pressure q in pounds per square foot for various building heights. For any structure under 30 ft in height, the 30 ft curve is used. The equation for this family of velocity-pressure curves is $q = K\left[\left(\frac{h}{30}\right)^{1/7} v\right]^2$, where K is a constant equal to 0.00256 when using standard air density at sea level, h is the building height in feet above the ground level, and v is wind velocity in miles per hour.

The wind load pattern is determined by the aerodynamic characteristics of the building. To determine the simulated wind loading, the velocity pressure q in the above equation is modified by applying a form factor. The form factors used by the laboratory are based on investigations and recommendations made by the American Society of Civil Engineers (Fig. 2).

Load combinations used to form part of the loading criteria are the same as those that have gained general acceptance in the design field. They are: (1) dead load plus full snow load, (2) dead load plus full wind load, (3) dead load

[1] Project Engineer, Environmental Division, U. S. Naval Civil Engineering Research Evaluation Laboratory, Port Hueneme, Calif.

plus full snow load plus $\frac{1}{2}$ the wind load and (4) dead load plus $\frac{1}{2}$ snow load plus full wind load.

For the last two combinations a 30 per cent increase in working unit stress is allowed.

All test loadings are taken to 180 per cent of the load specified by the criteria. In other words, a minimum safety factor of 1.8 is required.

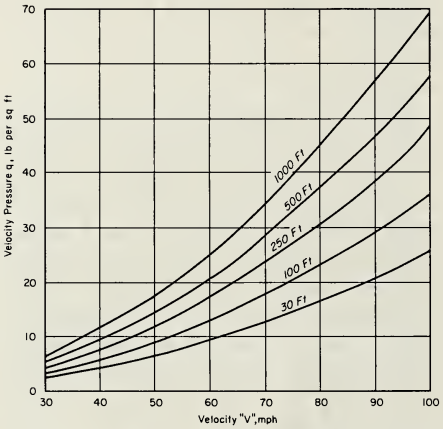


FIG. 1.—Velocity Pressure Curves Plotted from Equation $q = K\left[\left(\frac{h}{30}\right)^{1/7} v\right]^2$.

TEST FACILITIES AND PROCEDURES USED BY THE LABORATORY

The Laboratory's test procedures and facilities: have been refined as better methods became evident. Today, assembly line methods are used. A building undergoes three phases of testing: packaging and erection, weather, and structural. The test facilities are arranged to accomplish the three phases with a minimum of handling.

The Laboratory has two separate test facilities one for 40 by 100 ft buildings and one for 20 by 48 ft buildings. The 20 by 48 ft class has accounted for a major portion of the test work (Fig. 3). The test facility for 20 by 48 ft buildings is divided into three stations connected by a 20-ft wide I-beam track. All test work is done on a cart mounted on the track. The building once erected on the cart can be moved from station to station without dismantling.

At Station I erection of the building is accomplished. This is the first phase of the test procedure and the following features of the building are checked and studied: (1) weight and cube of the crated building, (2) package layout of building, and (3) erection procedure and number of manhours required to complete the erection.

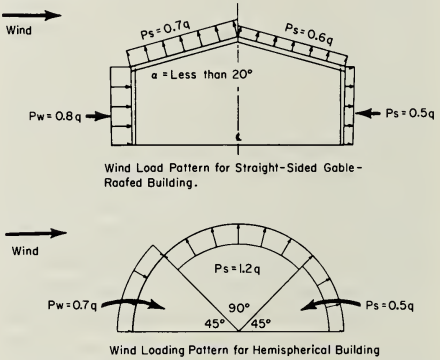


FIG. 2.—Form Factors Applied to the Velocity Pressure for Wind Load Distribution.

At Station II, weather testing takes place. The building is observed under a simulated natural rainfall of 2 in. per hr and a 35-mph wind-driven rain. An overhead sprinkler system simulates natural rain and a portable turbine spray duster provides the simulated wind-driven rain. To produce the wind-driven rain a perforated water pipe was installed in the air discharge throat of the spray duster. The normal procedure is to subject the building to the following weather tests: (1) natural rain falling at the rate of 2 in. per hr for a minimum of 30 min, and (2) wind-driven rain from the spray duster for a minimum of 6 passes per unit area.

After observations have been made at

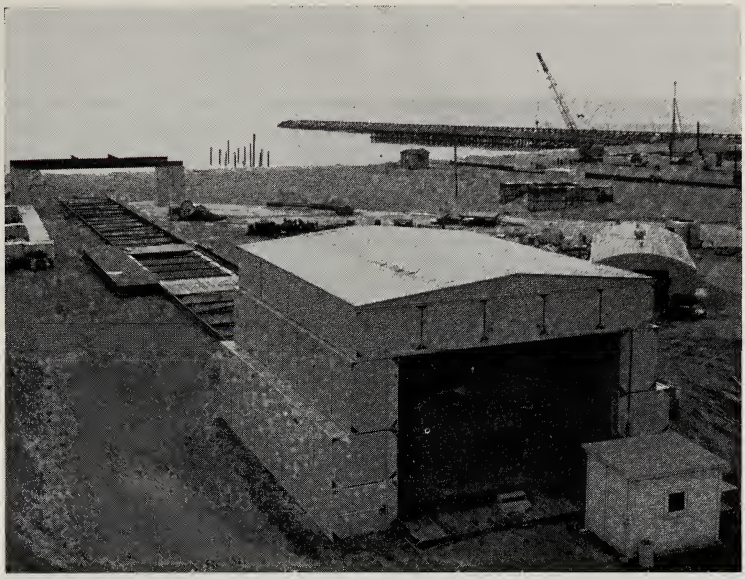


FIG. 3.—Complete Test Facilities for 20- by 48-ft Buildings. Structural Jig in Foreground and Weather Test Jig in Background.

FIG. 4.—Full 20 by 48-ft Stress Skin Type Building Rigged for Snow Load.

Station II, the building is moved to Station III where the structural testing is accomplished. The building and test cart are moved into a test jig constructed from Navy pontons. The pontoons form a jig having approximate dimensions of 42-ft length, 27-ft clear width, and 14-ft ceiling height above the deck of the cart.

Simulated loads on the building are produced by hydraulic cylinders. The cylinders are bolted to angles welded to the jig. The angles are set 8 ft on centers and have a 2-in. center to center hole spacing along the outstanding legs. This system provides an easy method for mounting the hydraulic cylinders in a pattern most suitable for the test. As many as 36 hydraulic cylinders have been used to test a completely assembled 48-ft building. It should be brought out, however, that only when the design utilizes a stressed-skin or frameless principle is it necessary to test a completely assembled building. Early in the test program it was determined that the end wall in a rigid frame or arch frame building adds very little rigidity to the structure except for a few feet near the end wall. Therefore, in the frame type building the laboratory tests only a typical 16-ft center section. The section selected is generally 5 frames in length as the frames are commonly on 4-ft modules. Concentrated loads from the hydraulic cylinders are spread by a whippletree system to produce a uniform load on the building (Fig. 4). This system is readily adjustable to insure load distribution. Wood pads, made from 2 by 4 material or plywood, are used to further distribute the load from the whippletrees to the building. The pads generally are tailor-made for each building.

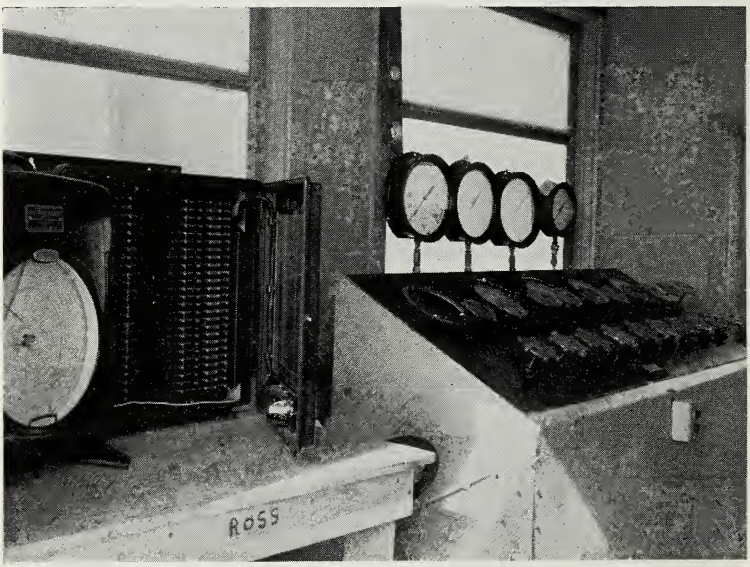


FIG. 5.—Hydraulic Control Panel to the Right and Strain Recording Instrument on the Left.

All cylinders are fed with hydraulic fluid from one central five-channel control panel. The fluid is pumped into the system by an electric motor-driven pump

capable of delivering a maximum pressure of 1500 psi. Pressure in each channel is indicated on a gage graduated in 10 psi increments through a range of 0 to 1500 psi. A manually operated valve controls each channel pressure (Fig. 5).

STRAIN MEASUREMENT

Strain at any predetermined point in the building is measured with an SR-4 strain gage. The type of gage most suitable for this application is a paper-backed gage which can be cemented to the member. A Baldwin A-5-1 gage, $\frac{1}{2}$ in. in length, 120-ohm resistance, is used for most applications. The number of gages used varies with each building and is dependent on the anticipated stress distribution. A few additional check gages are installed on adjacent frames in locations corresponding to those on the main strain-gaged frame.

The laboratory uses both manual and automatic recording instruments for measuring strain gage output. For manual operation the Baldwin type K or M strain indicator is used. These instruments are very suitable for portable use but become awkward when a large number of readings must be recorded. A Foxboro SR-4 pen type Scanner-Recorder is employed when strain measurements are required at many locations. This unit is an automatic recording type and with its companion switching unit has a capacity of 48 strain gages. The unit strain is measured in microinches per inch by both instruments. The measured unit strain is converted to unit stress by multiplying it by the modulus of elasticity of the material being tested.

FIG. 6.—Laboratory Developed Deflection Gages Used to Measure Deflection of Building Under Load.

DEFLECTION MEASUREMENT

Measurement of the deflection caused by the specified loading is accomplished with a deflectometer (Fig. 6). The deflectometer is a mechanical, dial type gage developed by the Laboratory. The

mechanism of the gage is very simple. It has as a main working part a spirally grooved spool around which a fine wire cable is wound. One end of the cable is attached to the building and the other end is counterweighted to take up the slack in the cable as the building deflects. A pointer rotates with the spool as deflection of the building occurs. The dial face of the deflectometer is calibrated in 0.1-in. increments; however, because of the large dial, it is easy to estimate the deflection to the nearest 0.01 in.

OTHER TEST FACILITIES

In addition to the test facility described for 20 by 48-ft buildings, the laboratory has facilities for testing 40 by 100-ft buildings and for testing floor sections of buildings.

The floor test jig consists of angles embedded in a concrete slab and adjustable pipe columns that can be arranged to duplicate the support points the floor has when installed in the building. The floor system is tested for a design loading 30 lb per sq ft. This loading is accomplished hydraulically. The strain and deflection measurements are made as previously described for testing the superstructure.

SUMMARY

In summary, assembly-line methods have enabled the Laboratory to test and evaluate 25 buildings in the 20 by 48-ft class and 5 buildings in the 40 by 100-ft class with a minimum amount of manpower. The normal working force consists of two field personnel and the project engineer. However, to duplicate actual field conditions the number of field personnel is increased during the erection phase.

The laboratory used the experience gained through the test program to develop a 20 by 48 ft, vertical wall, gable roof building. A main feature of the design is that framing members are fabricated from 18-gage steel with box-shaped columns and roof beams. This design fully complies with the design criteria and in addition provides an economical structure that is architecturally attractive both inside and out (Fig 7.).

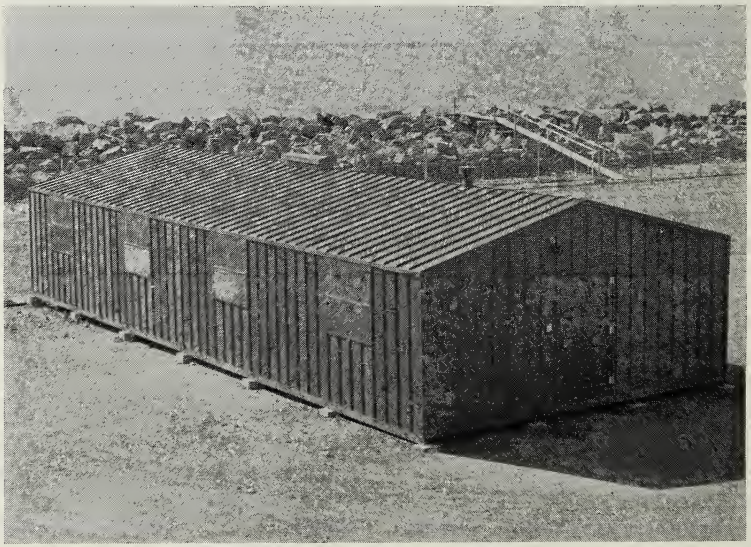


FIG. 7.—Straight-Sided, Steel "Hueneme" Building Developed by the Laboratory to Meet Military Design Criteria.

DISCUSSION

MR. ELLIOTT O. STEPHENSON.[1]—How does that design specification compare with the American Standard-ASA A58.1-Minimum Design Loads in Buildings and Other Structures?

MR. J. E. DYKINS (*author*).—The American standard specification differs from ours in that it calls for a minimum design for snow load of 20 lb per sq ft while we use 15 lb per sq ft. The wind also differs in magnitude and distribution.

MR. J. W. GUNTHER.[2]—Are the facilities there restricted to Naval testing?

MR. DYKINS.—Not necessarily. Everything we test, though, is initiated by the Bureau of Yards and Docks, and if somebody wants an item tested they have to start it there.

MR. LYMAN W. WOOD.[3]—Has anything been done in the way of testing under dynamic loading, such as might occur from earthquake?

MR. DYKINS.—Not in our particular section. I believe there is some work carried on in another section of the laboratory where project work involves dynamic load tests.

MR. ELI CZERNIAK.[4]—It was mentioned that some deflections were measured. What criteria is used for acceptance? How much deflection under actual live load, combined with wind and snow, will be accepted?

MR. DYKINS.—That is difficult to answer. In a lot of cases it is a matter of judgment. We normally try to limit it to no more than 2 in.

CHAIRMAN ROBERT F. LEGGET.[5]—The questioner puts his finger on one of the very critical problems in connection with the testing of complete structures. It is my hope that Committee E-6 in its further deliberations can do something about this. It is a subject upon which very little is known. I know of only one paper on allowable deflections in buildings. It is a paper just published by the Institution of Civil Engineers in London, with regard to the allowable settlements on large buildings, particularly with relation to settlements.[6]

MR. T. K. MAY.[7]—I rather doubt that deflections would be important in this type of building.

MR. DYKINS.—Actually that is why I said what I did in answering the previous question. We do not have plastered walls or ceilings so we are not too concerned with deflections. The buildings are used by the military services for both continental and advance bases.

---

[1] Engineer, American Iron and Steel Inst., New York, N. Y.

[2] Engineering Div., Union Wire Rope Corp., Kansas City, Mo.

[3] U. S. Forest Products Lab., Forest Service, U. S. Department of Agriculture, Madison, Wis.

[4] Design Engineering Supervisor, Fluor Corp., Ltd., East Los Angeles Branch., Calif.

[5] Director of Building Research, National Research Council of Canada, Division of Building Research, Ottawa, Ont., Canada.

[6] A. W. Skempton and D. H. McDonald, "Allowable Settlements of Buildings," *Proceedings*, Institution of Civil Engineers, Part 3, Vol. 5, No. 3, Dec., 1956, p. 727–784.

[7] Director of Technical Service, West Coast Lumbermen's Assn., Portland, Ore.

MR. F. M. ANDRUS.[8]—Did the rigid frame building have a roof diaphragm that transmitted the load to the end walls? Where there were end walls that may have influenced the building only near the end, was there any type of roof diaphragm that was capable of transmitting the lateral load to the end wall?

MR. DYKINS.—I recently tested a building of plywood construction. In this building the roof very significantly transmitted the load to the end walls. We have also tested other entire stress skin buildings in which the end walls were important.

MR. ANDRUS.—I would think so, because of the relative stiffness of the end wall compared to a frame, that the roof diaphragm would have more of an effect rather than just for a short distance.

MR. DYKINS.—It does in that type of building.

MR. W. R. SCHRIEVER.[9]—Could I ask if a design snow load of 15 lb per sq ft is used independent of the location of the building? Not only does 15 lb seem low, but it was mentioned that the building was specifically designed for northern locations. In the snow measurements which have been taken on the ground all over Canada snow loads up to 60 or 90 lb per sq ft have actually been measured.

MR. DYKINS.—As I said earlier, that is for a northern type building. The exact latitude at which you would change to a frigid or Arctic type building I do not know. For our Arctic building tests 60 lb load has been normally used. A northern type building has been erected at Fort Churchill for about $1\frac{1}{2}$ yr. Structurally it has been very satisfactory.

CHAIRMAN LEGGET.—I am beginning to think Canada must be in the frigid zone. It so happens that Churchill happens to be one of the places in Canada where we have very low snow loads.

[8] Principal Structural Engineer., Los Angeles County Engineers, Architectural Div., Los Angeles, Calif.

[9] Building Design Section, National Research Council of Canada, Division of Building Research, Ottawa, Ont., Canada.

FULL-SCALE TESTING OF PREFABRICATED MILITARY BUILDINGS

By Raymond F. Bartelmes[1]

The Engineer Research and Development Laboratories (ERDL), Corps of Engineers, Dept. of the Army, have been engaged in the design, testing, and evaluation of prefabricated buildings since late 1949. This paper deals with the full scale testing of these structures, primarily the tests of their structural frames.

All of the buildings are developed to meet a specific Army requirement which is defined and limited by the military characteristics outlined in an approved project. The military characteristics establish the size, configuration, installed equipment, design loads, either directly or by reference to climatic utilization, and request minimum packaged volume, weight, cost, and erection time. Building designs are accomplished by contract designers or, for the majority of cases, by the engineers and architects of the Buildings Design Branch, ERDL.

After a building is fabricated, it is tested against the requirements of the military characteristics as well as technical characteristics which may have been established separately for the class of building or the climatic zone. The gross and net weights and volumes are obtained and the building is erected for an initial timed erection and to check out ease of assembly of all parts, tolerances, and detail changes which may be necessary. In some cases the exterior cladding and interior insulation and liner are installed at this stage, if they form a part of the building, or structural tests are run after the roof sheets or panels have been installed. The completed building is dismantled for a timed disassembly. Components are examined for damage since the buildings must be demountable and re-erectable for a minimum of three erections without serious damage. A second erection is performed to obtain a more accurate indication of erection time.

The data obtained from all tests is used for comparison with data on existing buildings or standards. It is presented in a locally filed report or as an official ERDL report of wide interdepartmental circulation depending on interest known to exist.

Development work prior to 1953 was on 20 ft wide by 48 ft long buildings and smaller shelters. In these 20 by 48-ft buildings, combined live and dead loads were the most critical on the building's structural system, so the tests were confined to sandbag loading two or more bays of the floor and roof. The sandbags are placed in increment loads of less than design load, design load, and two or more increments up to safety factor loading[2] depending on the strain gage indications of criticality in any member. Readings are taken with an engineer's level of deflection gages which are previously attached to the top of floor beams and

[1] Chief, Building Design Branch, Civil Engineering Dept. U. S. Corps of Engineers, Research and Development Laboratories, Fort Belvoir, Va.

[2] 1.85 or 2.00 per AISI requirements for light gage steel or 1.5 for wood.

joists and to the underside of roof beams or trusses and purlins. Side sway readings of gages attached to the outside of columns of the test bays are taken with a transit.

The sandbag method of applying live load (Fig. 1) was chosen over a mechanical-hydraulic loading arrangement for several reasons:

1. It more nearly approximates actual conditions than where an equivalent point loading at one or two points on a member is substituted for the uniform load.

FIG. 1.—Placing Sand Bag Rack as Simulated Snow Load. Bags are Replaced With Heavier Bags for Increased Loading.

2. It is a cheaper method and requires no setup time other than for gages. In testing buildings of different widths, roof slope, and wall height, mechanical rigs require considerable modification for each building dimension change.

3. Loads are known to more certainty by simple checking of sand bag weights while it not so practical to check pressure gages frequently or to maintain an even pressure at all required points while making deflection and strain gage readings and for checking fatigue under prolonged loading.

SR-4 strain gages are applied to the framing members at principal high stress points taken from the design calculations and at intermediate points if the engineer desires to check for points of counterflexure (as in partially restrained columns which were designed as hinged). The strains are measured in microinches on a Baldwin SR-4 Type L strain indicator connected to four 20-channel switching units.

There is a strong tendency for test engineers to apply too many strain gages and obtain too many readings of doubtful value. He feels compelled to obtain sufficient readings to plot the moment curve, presumably to check the designers. As a result he may become bogged down in the mechanics of preparing and placing the great number of gages and obtaining readings. Mistakes are more apt to be made than in a simple setup of the absolute minimum of readings. Further, the elapsed time of the test becomes greater with incidental errors creeping in from strain gage drift due to variable sun heat and humidity changes.

Out of the 20-odd buildings of the 20 by 48-ft size tested up to mid year of 1951, only three were accepted as having generally conformed to the military characteristics and had passed their structural tests under static live loading. It now became necessary to determine whether these three buildings would withstand the required wind loadings. A study was made of all available data on the subject of wind effects on buildings. Among the more thorough treatments of the subject are studies on model testing, one being Thompson's "Protection of Small Buildings Against High-Velocity Winds,"[3] another Chien, Feng, Wang and Siao's "Wind-Tunnel Studies of Pressure Distribution of Elementary Building Forms,"[4] and Newmark's "Anal-

[3] R. A. Thompson, "Protection of Small Buildings against High Velocity Winds," *Bulletin No. 28*, University of Florida

[4] Chien, Feng, Wang, and Siao, "Wind Tunnel Studies of Pressure Distribution of Elementary Building Forms," *Master's thesis*, University of Iowa.

ysis and Design of Structures Subjected to Dynamic Loading."[5] A review of these studies by the author led to a conclusion that model tests in wind tunnels do not duplicate dynamic loads of a natural wind on a full scale building. Further, the behavior of the structure under static and slowly applied loads is different than the behavior of the structure under rapidly applied loads as occur in wind gusts buffeting the structure. It was decided, therefore, to subject the full size building to wind, either man-made or natural.

FIG. 2.—Buildings Erected for Wind Tests at Mount Washington. Instrument Shelter is Located Between Buildings.

WIND LOADING BY AIRPLANE

Various means of testing for wind loads were tried. The first attempt in this direction was to apply the full wind load called for directly to the full size building, which, at the time, was still 20 by 48 by 8 ft. A building was erected at Andrews Air Force Base near to an airplane hardstand. Pitot tubes and rear wall deflection gages were installed over 8 ft of the building length. Two B-26 aircraft were placed in predetermined positions to direct their wind against an anemometer and to correlate the wind velocities with the aircraft manifold pressures. The two planes were then relocated to corresponding positions to direct their propeller blast against the section of the building where 24 pitot tubes had been installed. The tubes were connected by plastic tubing to a manometer tube bank which had been placed in a pit under the floor of the building. The pit was used to shorten tubing runs and to provide protection to personnel and instruments if the building failed. One end of all U-tubes was connected to a manifold which in turn was connected through a valve to a pitot tube located in the free wind stream 6 ft in front of the building and also to a valved tube opening in the interior of the building. Deflection gages were attached to the lee eave of the building and were read through a transit located at the end of the building.

Due to wind turbulence, interference from natural wind, and other causes, the fluid in the manometer tubes fluctuated rapidly so that it was impossible to photograph a maximum condition at any wind velocity. Maximums were visually read and recorded for each tube for the differential between the interior pressure and points on the building surface and also the pressure differences between these points and the tube in front of the building.

TESTS AT MOUNT WASHINGTON

1951 Tests:

The tests at Andrews Field showed this method of wind loading to be impractical; therefore it was decided to test at Mount Washington, N. H., where winds in excess of 100 mph could be expected at frequent intervals. A site was selected at the Signal Corps station on the Horn halfway up the mountain. Since there were no level building sites, it was necessary to cut away the rock to provide a level area for the erection of a test building. A 20 by 48-ft steel panel frameless building was erected during the fall

[5] N. M. Newmark, "Analysis and Design of Structures Subjected to Dynamic Loading."

of 1951. No instrumentation was installed, although wind data were obtained from the nearby Signal Corps station. Cable guys were attached from the building to continuous concrete deadmen on each side of the building. These guys were slackened on the windward side during observations in wind of up to 60 to 70 mph since the building was required to withstand 60 mph winds without guys. Vibrations in the building were observed to start at approximately 40 mph; at 80 mph a frequency resonance occurred which caused the roof to suddenly lift off the top plate although bolted down with hook bolts. After bolting the top plate and base angle to the wall panels with through bolts, this no longer occurred, although a pronounced resonance peak continued to occur at approximately 80 mph.

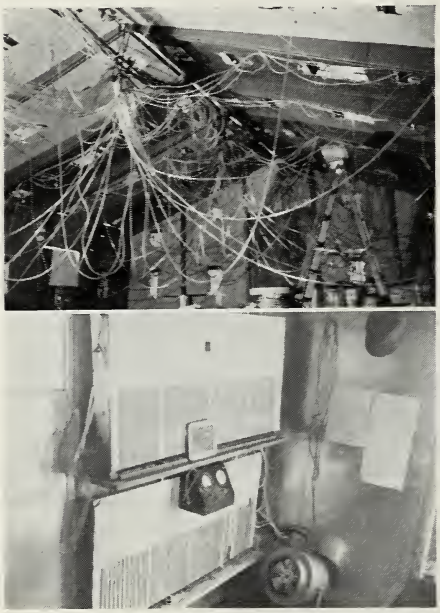


FIG. 3.—Top—Plastic Tubing Attached to Pitot Tubes Connect to Manometer Banks (below).

1952 Tests:

For the next season of testing it was decided to instrument two test buildings, both of which were rigid frame types each having light structural frames at 4 ft on center. The first building was dismantled and a second site was cut out of the rock and the two buildings were erected (Fig. 2). A number of pitot tubes

FIG. 4.—Top—Electronic Wind Probe. Bottom—Wind Velocity Recording Equipment.

were installed in the buildings at various locations based on the Iowa wind tunnel studies. Plastic tubing (Fig. 3) was run from each pitot tube through conduit boxes to a manometer bank in a small instrument shelter located between the two buildings. Strain gages were installed on frames and building panels and connected by wiring to the automatic recorder in the observation shelter. The anemometer indicator was also located in the same small shelter.

In preparation for the tests at Mount Washington, the Reed Instrument Co. manufactured a 150-channel automatic strain gage amplifier and recorder. An instantaneous reading electronic anemometer was procured from the Hastings Instrument Co. The strain gage recorder was found to be defective and was sent back to the company for modifications. No opportunity was found to recheck the instrument before its use in the tests where it again proved to be a source of trouble and readings were considered to be inaccurate. The wind probe of the electronic anemometer (Fig. 4) failed on one occasion but was repaired by a company representative.

All of the work in getting ready for the tests was done under the most trying conditions. The location is very remote and transportation of materials and the men to and from the site several miles up the mountain was very time consuming. On numerous occasions, the wind was of too high a velocity to permit travel up the mountain and necessitated those working to seek shelter in the Signal Corps station. Building parts that were not weighted down with heavy boulders blew away; and the plywood floor on one building that was partially erected and left over night, blew away taking steel joists with it. Framing bents were knocked down and distorted. Another building had to be shipped in before construction could be completed. With the many delays it was January 1953 before the buildings were erected and instrumentation completed. By this time it had become too hazardous to make the trip up the mountain, so testing was deferred until March. During March and April, readings were made for various wind velocities in an attempt to correlate wind pressure with velocity and the effects on the building.

Although numerous tape strips were run on the strain gage recorder and pressures and velocities taken, the data could not be accurately correlated because of the impossibility of establishing zero readings on the strain gages at periods of no wind and have these readings as a base when a high velocity wind was being recorded. Ice formation in the pitot tubes also interfered with the pressure readings. The voluminous data have not been analyzed at this writing because the several people connected with this work have left ERDL. However, it is planned to engage a qualified laboratory or individual to make a study of the data.

Arctic Buildings

While the wind loadings for the general purpose type buildings have been established at 60 to 70 mph with gusts to 100 mph, the arctic shelter project card stated that the building should withstand sustained winds of 100 mph and gusts of 125 mph. The U. S. Forest Products Laboratory had tested a 4 by 8-ft wall panel of this building in bending and found it to withstand 265 lb per sq ft. The laboratory also analyzed the design and predicted it would not overturn or fail under winds of 125 mph, but might slide. At our test site at Fort Churchill on Hudson Bay, the maximum wind on the building had been 55 mph. Not wishing to go through the difficulties of another Mount Washington test, we decided to return to conventional methods of wind testing and apply an "equiv-

alent" static load. Holes were drilled halfway up the walls and several cables were passed through and attached to channels on the far side of the building. Dynamometers were installed in each line which then passed horizontally over a frame and down at an angle to a ratchet hoist which was anchored to heavy steel stakes driven in the ground. No attempt was made to duplicate the positive and negative loads on the roof. The building successfully withstood the 125 mph static equivalent load which was considered to be decidedly more severe than the dynamic short term loads of natural wind. Sliding was prevented by blocking against the floor panels.

FIG. 5.—Jib Boom Cranes Were Used to Raise Sand Bags on Racks to Roof of 22-ft Wall Shop Building.

TESTING THE BUILDING SYSTEM

Nine Buildings to Test:

Beginning in late 1953, the ERD Laboratory has been developing a prefabricated building system composed of a number of interrelated buildings with a maximum number of interchangeable parts. Three design contracts were awarded: a building manufacturer who retained a research organization to do the design which was based on nestable Z sections; a bar joist manufacturer who engaged an architect-engineer firm and utilized his commercial bar joist and studs insofar as possible in the design; and a development contractor associated with a well-known structural engineering firm who utilized gage metal box sections for their members. Each of the three designs of the three basic buildings appeared to warrant the fabrication of test buildings of minimum lengths. This meant that nine buildings had to be tested, and six of these were considerably larger than the single 20 by 48-ft buildings previously tested.

Snow Loads:

The placement of a sandbag simulated snow load on the 22-ft wall height shop buildings was accomplished by installing a 20-ft jib boom on the end of a regular 50-ft crane boom and lifting a series of racks to position on the roof (Fig. 5). Sand bags of selected weights and numbers were tied to the cross members of the racks to give the roof loading increment desired. The usual test was to load a 20-ft length of building which had been disconnected from adjacent building bays. The racks were removed and replaced with heavier sand bags as increased loadings were applied.

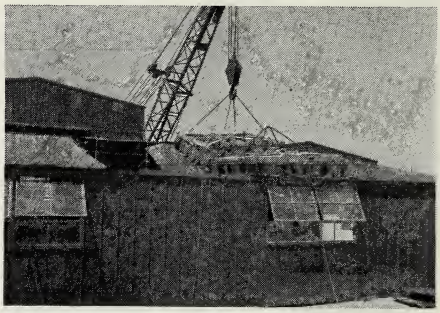


FIG. 6.—Roof Collapsed as Last Rack was Placed at 23 lb per sq ft.

Failure of Box Member:

Because of the number of buildings to be tested, an attempt was first made to start the loadings at design load and

immediately jump to the safety factor load. The roof of the 20 by 50-ft box frame barracks collapsed under this second load (Fig. 6). Therefore, it was necessary to replace the damaged members and repeat the tests using a smaller increment of increased load. The roof beam again collapsed, but an accurate loading was known and strain gage readings were made at a load slightly under the failure load. These readings showed relatively low stresses, so the design calculations were reviewed and found to be in apparent good order except for an assumption that the flanges of the beam

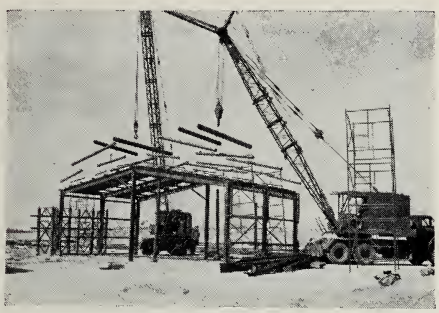


FIG. 7.—Simulated Wind Load Applied to 20-ft Bay of 40-ft Wide Warehouse.

were a single thickness of metal whereas they were actually formed by spot welding two gage metal channels together. In addition, no tests had been run by the designer on single members to determine a buckling factor. After two failures in the building, a roof beam was loaded as a simple beam and again failed by web and flange buckling at the same loading.

Wind Load Tests:

Wind load tests were performed on the building by rigging pipe and cable harnesses to load purlin and girt locations on two adjacent framing bents. Uplift on the roof was made by pulling through a six-part line from a crane boom. Positive and negative pressures were applied to windward and leeward wall columns respectively, by pulling with anchored ratchet hoists on cable lines and harnesses attached to the columns. The box section roof beam again failed at less than safety factor load. Since this beam was used in the two larger buildings of the system, tests of these buildings were considered as fruitless as well as dangerous, so they were not undertaken.

Simulated wind loads were applied to four of the remaining six buildings. The two shop buildings were too tall to utilize the crane booms for uplift loading on the roof. However, the 40-ft wide warehouse buildings of 13-ft wall height were tested. Two cranes were required (Fig. 7) to furnish uplift to the rather large expanse of roof. Horizontal cables pulling through harnesses again provided wall loading. All loads were read with new dynamometers placed in key locations in the harnesses. Strain gages previously placed on critical sections were read for each increment of loading.

SUMMARY

The failure of the box member design has been previously described. The Z-section system passed the structural tests except that bolt holes in web connections of the primary framing had to be reinforced with washers welded on as bossings to eliminate bearing failure at these points. This system was eliminated because of complex design and difficulty of erection. The third system of commercial bar joist members also passed without structural failure of members but had joint failures. This system was selected as requiring the least design changes and was the simplest and most rugged of the three designs.

While a moderate amount of testing of a building is desirable to check out architectural features, erection and disassembly time, and for packaging data, it is

now believed that novel designs, particularly in light gage metals, should first be structurally tested as a single bent. If bracing the bent during tests is a problem, then three bents and secondary framing should be the maximum required. This will save considerable time and expense of procuring, erecting, and testing buildings which are not structurally adequate or susceptible to minor modification to make them adequate.

In order to state that a light frame prefabricated building under consideration for adoption by the military meets the structural requirements, it is necessary to impose these loads on prototype buildings. Snow loads can be imposed by placing sand bags or hanging dead weight from the roof members. Wind loads cannot readily be duplicated and "equivalent" static loads must be applied to the structure by mechanical means.

STRUCTURAL TEST OF A HOUSE UNDER SIMULATED WIND AND SNOW LOADS

By D. B. Dorey[1] and W. R. Schriever[1]

House structures, in contrast to industrial buildings and other larger structures, do not generally lend themselves to the normal methods of structural design. For this reason the design of wood frame houses in Canada, as well as in other countries with similar climatic conditions, has developed largely through experience or tradition.

It is often thought that conventional house construction is overdesigned and that substantial economies could be achieved if a means could be found of evaluating the structural strength and rigidity of a completed house. This need for evaluating the structural sufficiency of a house design has become more pronounced with the introduction of new materials and new methods of combining both old and new materials in house construction. In some cases, a new construction can be compared structurally with conventional construction with some success, but in other cases there is no direct basis of comparison. Even when a comparison is possible, there remains the question of overdesign as it is frequently not known if conventional construction is overdesigned. The structural assessment of house designs must, therefore, be based in part upon information obtained from full-scale house tests in which simulated live loads are applied to a house.

[1] Building Design Section, National Research Council of Canada, Division of Building Research, Ottawa, Ont.

Shortly after the formation of the Division of Building Research (DBR) of the National Research Council of Canada, in Ottawa in 1947, it was decided to begin a research project on structural testing of full-scale houses. This paper describes the first such test, conducted by the Division in the summer of 1954.

Work on this project was begun with a literature survey of pertinent work by other research organizations. It was found that work in this field had mainly been done in the United States by the Naval Civil Engineering Research and Evaluation Laboratory at Port Hueneme, Calif.; in Great Britain by the British Building Research Station; and in South Africa by the National Building Research Institute.

DBR Experimental House

The house chosen for the structural test was one that had been built in 1948 for experimental purposes but not particularly for a structural test. The floor plan and cross-section of this house are shown in Figs. 1 and 2, respectively. The house was a basementless, one-story, wood-frame dwelling about 36 ft long and 24 ft wide. It had three bedrooms, a living room, a kitchen, a bathroom, and a utility room.

The foundation walls were of concrete blocks and extended to a depth of 3 or 4 ft below grade to bedrock. Sill plates, 2 by 6 in., were fastened to the top of the foundation wall with anchor bolts.

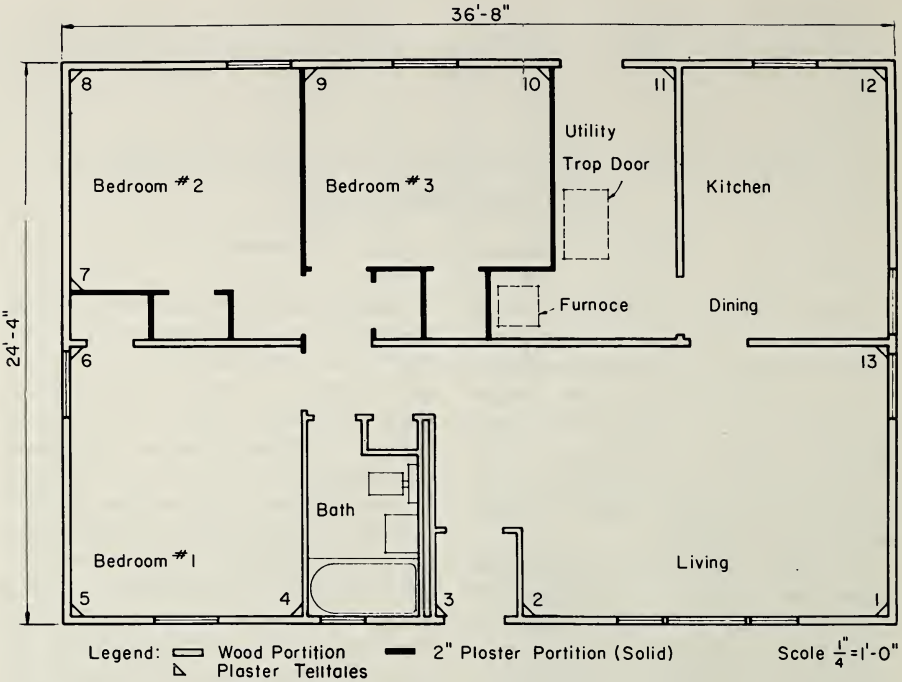


FIG. 1.—Floor Plan of Experimental House No. 2.

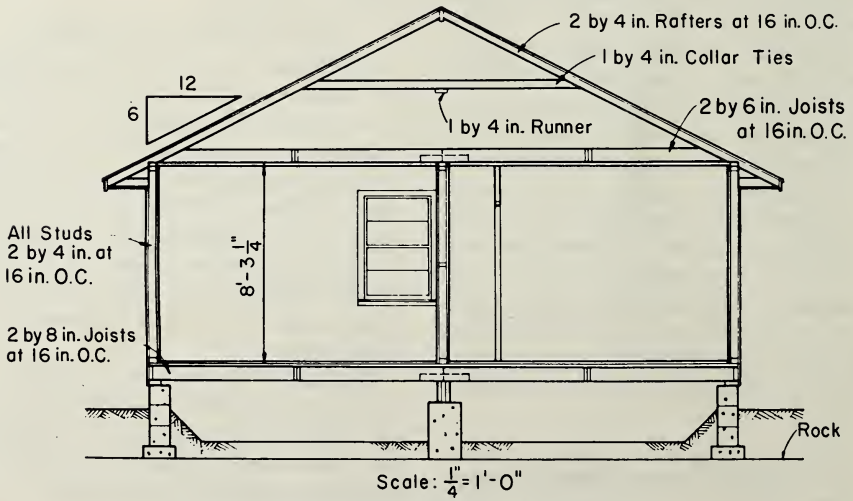


FIG. 2.—Cross-Section of Experimental House No. 2.

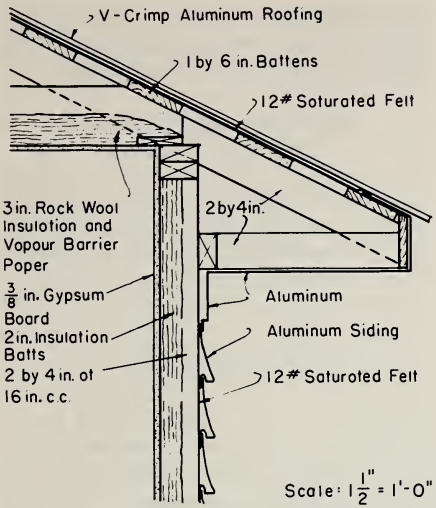

V-Crimp Aluminum Roofing
1 by 6 in. Battens
12# Saturated Felt
3 in. Rock Wool Insulation and Vapour Barrier Paper
2 by 4 in.
3/8 in. Gypsum Board
2 in. Insulation Batts
2 by 4 in. at 16 in. c.c.
Aluminum
Aluminum Siding
12# Saturated Felt
Scale: 1 1/2" = 1'-0"

FIG. 3.—Typical Wall Section.

The floor consisted of diagonal subflooring, $\frac{13}{16}$ in. thick, covered generally by hardwood flooring $\frac{3}{4}$ in. thick.

The exterior walls were constructed of 2 by 4-in. wood studding placed on 16-in. centers, and braced by 1 by 4-in. diagonal members let into the studs at each corner of the house. Horizontal 2 by 4-in. girths were placed between the studs at the midheight of the studs. The studding was covered on the outside with 12-lb asphalt-saturated felt paper, over which interlocking aluminum siding, $6\frac{7}{8}$ in. in width, was nailed to the studs (Fig. 3). Figure 4 shows part of the unclad framing and the aluminum siding being applied to the wall studs over the saturated felt.

An important and unusual feature of

FIG. 4.—View of Exterior Wall during Construction Showing Framing Details and Aluminum Siding being Applied.

Floor joists, 2 by 8 in., 16 in. on centers, spanned each half width of the house and were supported at the center of the house by a wooden beam made up of 2 by 10-in. planks, running lengthwise through the center of the house. This beam was supported in turn by the end walls of the concrete block foundation and by piers at approximately the third points of its span.

the construction of the exterior walls was the omission of exterior sheathing. Exterior sheathing normally contributes to the strength and stiffness of conventional exterior wall construction; it was not included in this structure so that a study could be made of the effect of such an omission.

The roof framing consisted of 2 by 4-in. rafters placed on 16-in. centers on

a 6 in 12 slope with 1 by 4-in. collar ties for each pair of rafters. The collar ties were fastened to each other at midspan by a 1 by 4-in. longitudinal member nailed to their underside.

The aluminum roofing was supported by 1 by 6-in. roofing boards which were nailed to the top of the rafters. These boards were spaced approximately 6 in. apart and were covered by 12-lb asphalt-saturated felt. V-crimp aluminum roofing, 30 in. wide, and running the full length of the rafters, was applied parallel to the rafters, over the asphalt-saturated felt. The aluminum sheets overlapped 7 in. along their sides and were double V-crimped at the joints.

The ceiling construction consisted of 2 by 6-in. joists with a 12-ft span, supported near the center of the house by a partition wall. The construction was done according to normal construction practice and does not represent above average workmanship. For example, the lower ends of the rafters were not all nailed directly to the joists at the wall plates, and thus, at some points, the ceiling joists were merely toe-nailed to the top wall plate. The interior finish of the ceiling was gypsum wallboard $\frac{3}{8}$-in. thick.

The central load-bearing partition and most of the other partitions were constructed of 2 by 4-in. studs placed on 16-in. centers. Three partition walls and some closet walls (Fig. 1), however, represented another experimental feature of the house consisting of 2-in. solid plaster walls. Figure 1 also shows the plaster telltales which were installed in a number of interior corners at ceiling level to observe damage resulting from loading. The plaster telltales were made by removing the interior paint from the gypsum wallboard over an area of several square inches on each of the three intersecting planes in each corner and by replacing the paint with returned layers of gypsum plaster to give a better indication of harmful deformations at the junction of the intersecting planes.

PURPOSE AND SCOPE OF STRUCTURAL TESTS

Structural tests that may be conducted on the various elements, components (partial assemblies), and complete assemblies of domestic dwellings are all interrelated. Structural tests on elements such as joists, rafters, studs, or on components such as floors, walls, or roofs, can reveal the strength and stiffness of the specimens but may not always give a proper indication of the strength and rigidity of the complete dwelling, assembled from these components or elements. Full-scale dwellings must be tested, therefore, if a useful attempt is to be made in correlating the results of such investigations. The testing of reduced-scale models would be difficult because of various scale effects, particularly with regard to nailing.

The two main purposes of the structural test on this house were (1) to obtain information on the strength and stiffness of a full-scale single-story house without exterior sheathing, and (2) to obtain experience in full-scale testing and in evaluating the strength of house frames.

A further reason for the test was the hope of obtaining information which would be useful in writing performance requirements for houses in Canada which could be used in connection with the acceptance of new types of construction.

The performance was to be evaluated on the basis of deformation and strength.

Deformations of the load-supporting members under design live load may, and often do, cause unsightly cracks in walls and ceilings. The possibility of such damage seemed particularly real in the house tested, as it had no exterior sheath-

ing. To check this it was decided to apply first the design loadings of the National Building Code of Canada (1953) for wind and snow and to observe deformations and possible damage under these loads.

The strength of the house was to be tested by loading beyond the design live loads to the point of failure. Since, however, it was not the intention to damage the test house beyond the state of possible repair, it was decided to carry the loading only to a point of relatively minor damage.

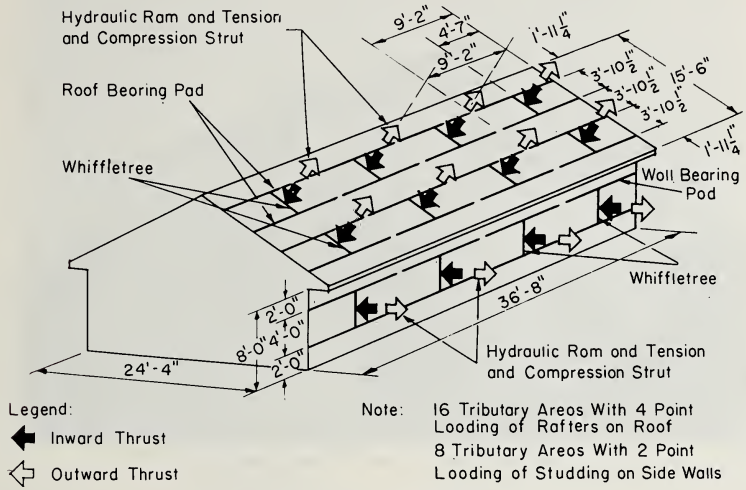


FIG. 5.—Points of Loading.

PREPARATIONS FOR TESTING EXPERIMENTAL HOUSE

Choice of a Loading System:

One of the first problems in the planning for this test was the selection of a loading method. A review of methods used by other research organizations revealed the two most promising methods. The first uses steel cables anchored to the ground at one end and directed over columns through a loading mechanism and a load-measuring device to load-distributing pads on the roof or side wall. The second method makes use of a reaction framework over the test house, inward and outward thrusts being applied by double-acting hydraulic rams.

The first method has the advantage of simplicity of assembly but is not readily adaptable for applying outward thrusts to moderately sloping roof surfaces and has some other disadvantages.

The second method is more complicated to design and assemble but may be justified if the facilities are available. With a rigid reaction framework spanning the test house, simultaneous inward or outward thrusts may be applied and controlled from a central position without altering the test house very much.

After careful consideration, and since the Division had an available stock of Bailey bridging components which could easily be adapted to use, it was decided that the reaction framework combined with hydraulic tension jacks to apply the loads should be used.

Another question that arose during the planning of this test was the choice of the

degree of load distribution to be achieved on the walls and roof. Maximum shears or bending moments could be duplicated in the members by concentrated loads, or a system of loading could be used that would approach uniform loading of the structure as a whole and produce the over-all effect of the uniform loadings assumed in the design. The latter system of loading was decided upon. The roof and side wall loading distribution is shown in Fig. 5.

Figure 5 also shows that the loading system was designed for the various loads to be applied at right angles to the roof and side walls. This system represents the normal wind loading condition but does not represent the correct system for snow loading which should be vertical (loads in pounds per square foot of horizontally projected area). Since, however, the test house did not have exterior sheathing to contribute strength against racking loads, it was originally thought that the wind loads would have the greatest influence on the interior finishes. As the additional work and expense involved in providing a loading system that could be used to apply loads either vertically or at right angles to the roof would have been considerable, it was decided to provide a system that would apply loads at right angles to the roof only and disregard the component of the snow loading parallel with the slope of the roof.

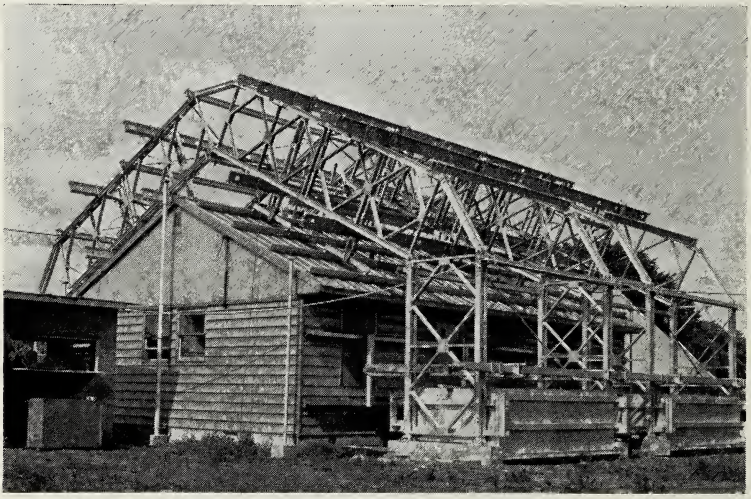


FIG. 6.—General View of Test Assembly.

Reaction Framework and Loading Members:

The reaction framework spanning over the top of the test house was composed mainly of Bailey bridging, with additional specially designed components. This framework was supported by special foundation pads and weighted down by large boxes filled with crushed stone. The erection of the steel work was done by an outside firm with the help of a mobile crane.

Each roof slope was divided into 16 equal tributary areas of loading and each side wall into 8 areas, as indicated in Fig. 5. Hardwood loading pads were installed on the side walls and the roof and bolted to the 4-in. I-beams. Sponge rubber $\frac{1}{8}$

in. thick was used between the surface of the roof and the hardwood to provide a compressible medium between the two surfaces. On the roof the pads were bonded by glue to the aluminum sheeting so that both an upward and downward force might be applied. Later during the test a partial breakdown in the glue bond occurred, making it necessary to fasten the pads by wires to the tension and compression struts. Figure 6 shows a general view of the test assembly.

Deflection Measuring Apparatus:

A system of pulleys and wires was used inside the test house to measure the deformations under load. This is a convenient system to measure in a central location deformations at widely separated points of a structure. It can be

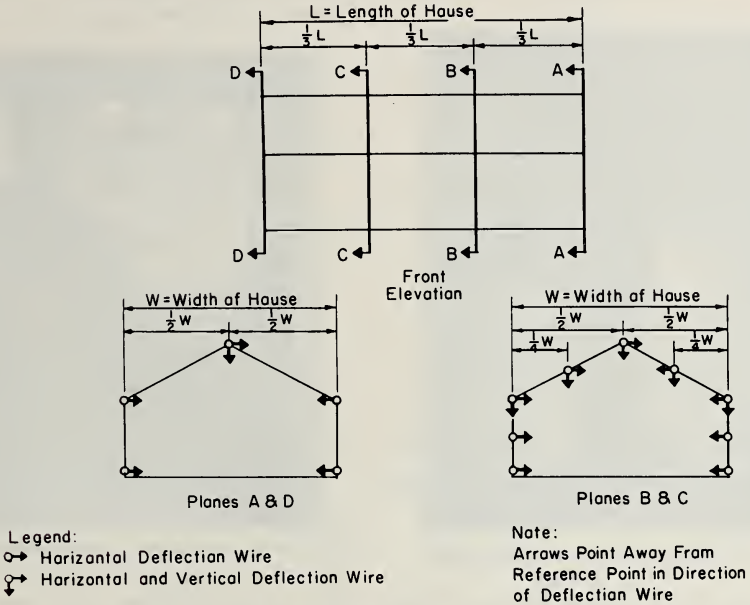


FIG. 7.—Planes of Deformation.

rafters through the roofing. The side wall loading pads were fastened to each stud with steel wire through the wall.

Pairs of loading pads were then connected at their midpoint to a cross-whiffletree which in turn was connected by a pin joint at its center point to a tension and compression strut. Each of the 24 tension and compression struts were restricted to unidirectional movement at right angles to the plane being loaded. Hydraulic rams were later mounted between the channels of the used when the deformations are expected to be large enough to be read without magnification.

Piano wire, 0.010 in. in diameter, running over a system of $2\frac{1}{2}$-in. low-friction aircraft pulleys, was installed to register horizontal and vertical movements on four deformation planes, one at each of the third points in the interior and one at each end wall on the exterior as shown in Fig. 7. The pulleys, over which the wires are running, must not be affected by the movements of the house

during the test. In this case the chimney, which is near the center of the house, was used to support the deflection apparatus inside the house and all connections between the chimney and the house were removed.

Two additional wires were installed on one of the end-wall planes to indicate vertical and endwise movement at the ridge-board level of the test house, and one wire to indicate whether the deflection apparatus had been disturbed. This made a total of 43 wires.

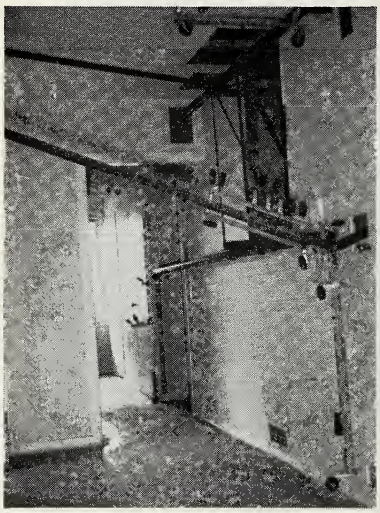


FIG. 8.—Interior Deflection Apparatus.

All the wires leading from the various points in the house were brought through one window opening in the end wall to a deflection board in the instrument hut. Figure 8 shows the deflection apparatus in the living room and hallway and the way in which the system was attached to the chimney through the larger opening made in the partition wall. Figure 9 shows the termination of the 43 wires on the deflection board, each wire being tensioned by a 1-lb weight, the lower edge of which was used in marking successive deflection readings on the board. The instrument hut also housed the hydraulic console unit and the power supply transformers.

Loading Equipment:

The hydraulic equipment for this test included 24 tension jacks, an air accumulator, a console unit, and special lines and fittings.

All hydraulic rams were calibrated for hydraulic pressures ranging from 0 to 2000 psi before the test. The connections to the hydraulic rams on the struts and Bailey transoms were so designed that the direction of the force exerted by the hydraulic rams could be reversed, thus making it possible to apply either a push or a pull to the area of loading.

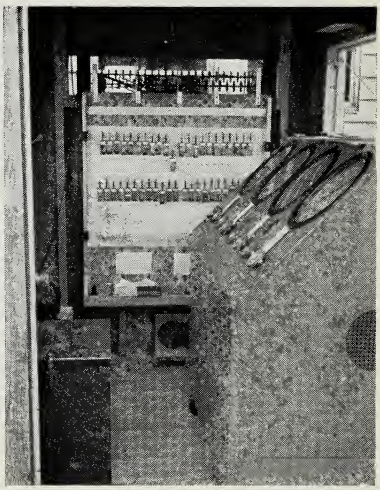
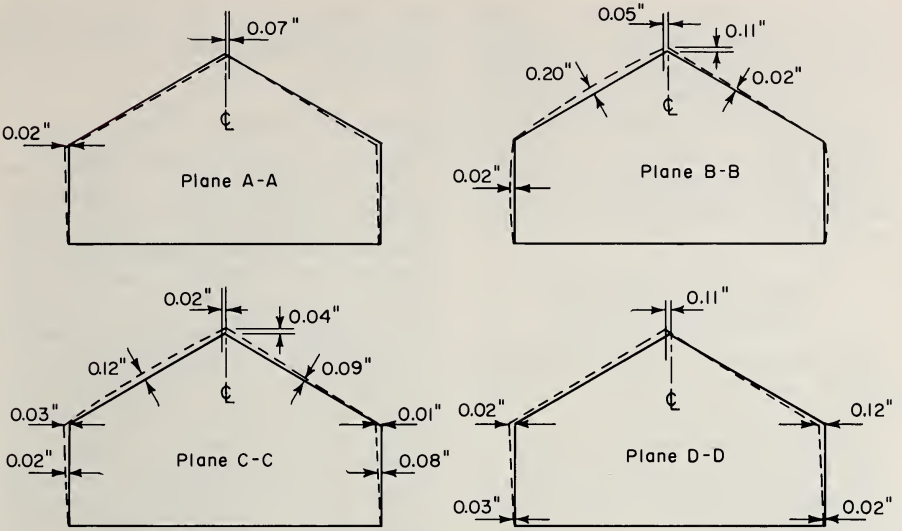
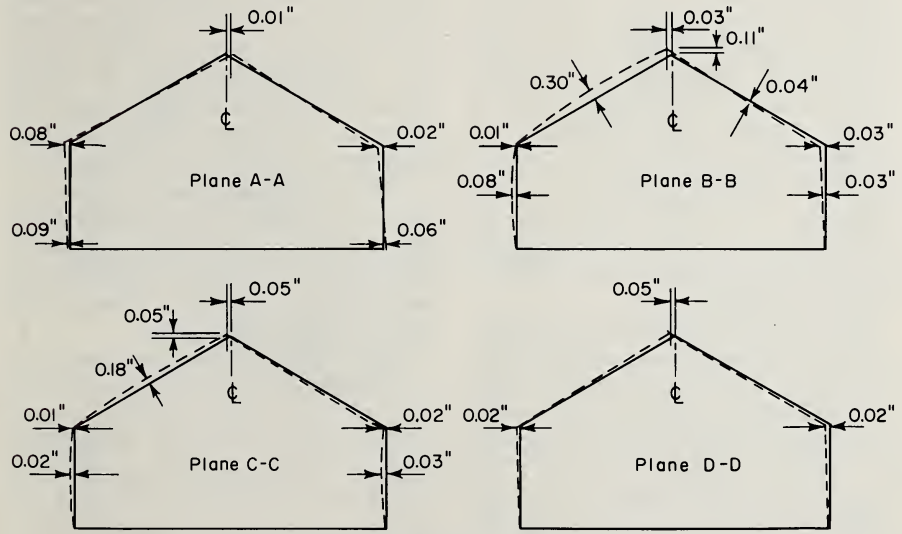


FIG. 9.—General View of Instrument House.

The rams were operated from the console with four pressure channels, each of which could be operated at a different hydraulic pressure, one for each of the four surfaces of the test house. Four rams of equal thrust capacity were installed on each of the side walls, and eight of equal thrust capacity on each of



(*a*) Wind Velocity 90 mph.



(*b*) Wind Velocity 120 mph.

———— Undistorted Frame
— — — Distorted Frame

FIG. 10.—Deformations due to Wind Loads (Internal Suction).

NOTES.—Refer to Figs. 5 and 7 for loading and deflection points. Deformations are not drawn to scale.

Wind Direction: From the right.

the two roof slopes. Each of the four pressure channels had its own pressure gage and operating valves in the console. To facilitate accurate loading an air accumulator, which could be charged with a booster pump to pressures in accord with the requirements of the hydraulic rams, was connected to the console unit.

STRUCTURAL TESTING OF EXPERIMENTAL HOUSE

The sequence of loading phases for this test was from the least to the most severe conditions as follows:

(*a*) Wind loads: internal suction,

(*b*) Wind loads: internal pressure,

(*c*) One-half design snow load plus wind loads, and

(*d*) Snow loads.

According to the 1953 edition of the National Building Code of Canada, 90 mph is the design wind velocity for the Ottawa area and 120 mph is the highest wind velocity that might reasonably be expected in the populated areas of Canada. It was decided, therefore, that the wind loads would first be carried to the design velocity of 90 mph and maintained for 1 hr, then released and reapplied up to a velocity of 120 mph for both an internal suction and an internal pressure condition.

There was some doubt as to what percentage of the design snow loading should be used for the combined loading condition which is not stipulated in the National Building Code. It was decided that the maximum amount of snow which might be expected to remain on the roof under high wind velocities would be not more than $\frac{1}{2}$ the design snow loading.

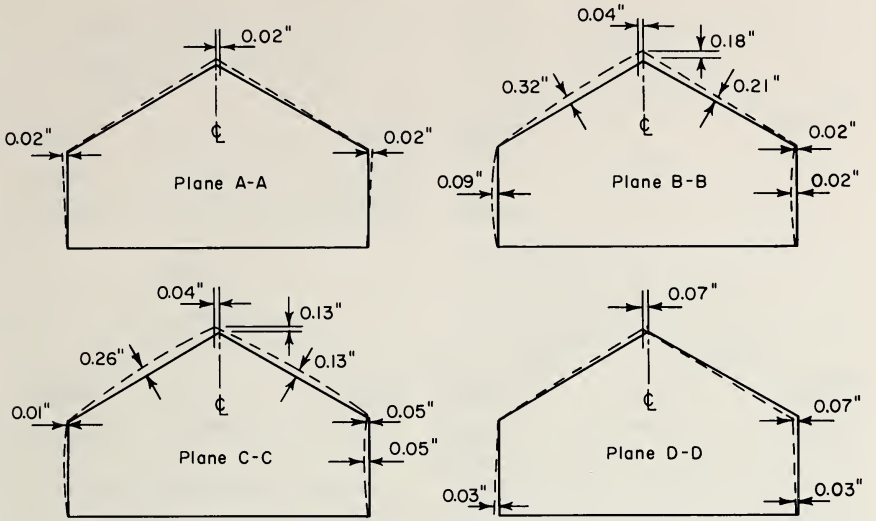
The magnitude of the design snow loading was arrived at by direct computation from the information given in the National Building Code (1953). The snow loads applied during the test were the components of the vertical loads acting at right angles to the plane of the roof.

Wind Loads—Internal Suction:

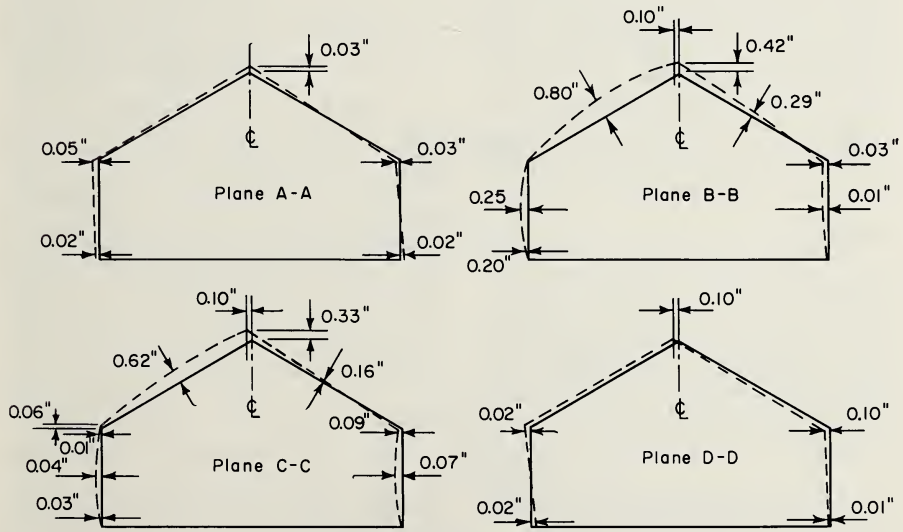
Internal suction (windows open on leeward side only) was the first wind loading condition applied. This condition is less severe than that producing internal pressure, as it results in smaller loads except on the windward wall. The loads were applied in increments of pressure corresponding to increases in velocity of 10-mph, beginning at 70 mph and ending at the design velocity of 90 mph. The loads were maintained for $\frac{1}{2}$ hr, and the loads for a velocity of 90 mph were maintained for 1 hr. Deformations were recorded on the deflection chart in the instrument hut before and after each increment of loading and after complete unloading. A visual inspection was made of each of the 13 plaster telltales before and after each loading increment to see if any cracks had developed. No cracks appeared during this loading phase. Deformations resulting from the first phase of loading are shown in Fig. 10 (*a*).

The next loading phase was a continuation of the loading schedule given above up to loads corresponding to a wind velocity of 120 mph. Each wind load was sustained for 1 hr after a 90-mph wind velocity loading was reached.

After reapplying pressures equivalent to a 90 mph wind velocity a crack in telltale No. 5 was discovered, and toward the end of the 1-hr period of sustained loading, a second crack became visible in telltale No. 9. All loads were then increased to pressures corresponding to 100, 110, and finally 120 mph and maintained for 1 hr in each case. During the 120-mph loading, cracks appeared in two more telltales, Nos. 6 and 10. The deformations resulting from this phase of loading are shown in Fig. 10 (*b*).



(*a*) Wind Velocity 90 mph.



(*b*) Wind Velocity 120 mph.

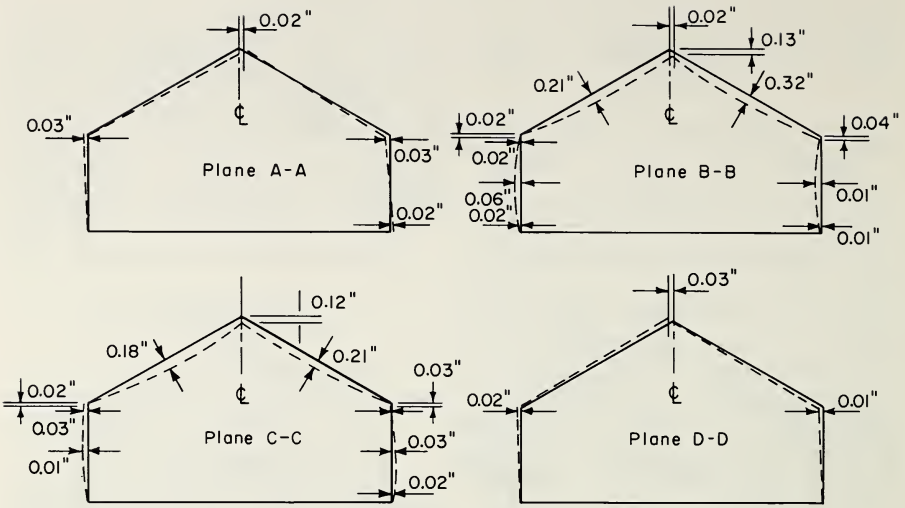
———— Undistorted Frame

— — — Distorted Frame

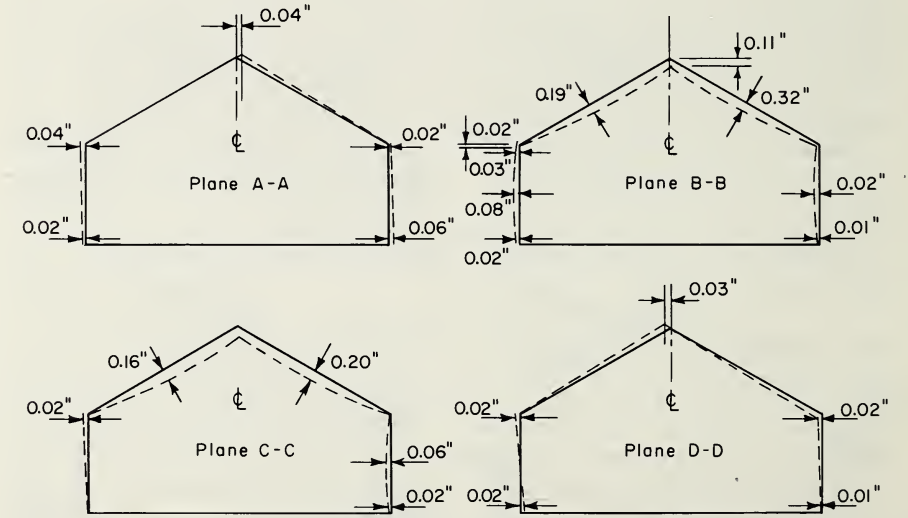
FIG. 11.—Deformations due to Wind Loads (Internal Pressure).

NOTES.—Refer to Figs. 5 and 7 for loading and deflection points. Deformations are not drawn to scale.

Wind Direction: From the right.



(*a*) Wind Velocity 90 mph.



(*b*) Wind Velocity 120 mph.

———— Undistorted Frame

— — — Distorted Frame

FIG. 12.—Deformations due to ½ Design Snow Load plus Wind Loads.

NOTES.—Refer to Figs. 5 and 7 for loading and deflection points Deformations are not drawn to scale.

Wind Direction: From the right.

At the end of 120 mph sustained loading, all pressures on the console were released in increments of one-third of the maximum pressures.

Wind Loads—Internal Pressure:

The next wind loading condition applied was that producing internal pressure (windows open on windward side only).

Loadings were again applied in increments of pressure corresponding to increases in velocity of 10 mph starting at a velocity of 70 mph and ending at the design wind velocity of 90 mph. At loads corresponding to a velocity of 80 mph, a new crack became visible in telltale No. 6. The 90-mph load was maintained for 1 hr, but no further cracks appeared in the plaster telltales. Then pressures were released in three steps to zero. The deformations resulting from this loading phase are shown in Fig. 11 (*a*).

The next phase was a continuation of this loading up to a wind velocity of 120 mph.

During the process of increasing and maintaining loads to 90, 100, and 110 mph, no further cracks appeared, but during the 1-hr period of sustained loading for a wind velocity of 120 mph, a crack which appeared in telltale No. 5 earlier in the test extended noticeably. At the end of the loading period, all loads were released to zero. The deformations resulting from this phase of loading are shown in Fig. 11 (*b*).

Up to this point in the test, the roof loads were in the form of an outward thrust at each of the loading points. Having completed the wind loading schedule, the next phase of loading in the direction of increasing severity of loading was that of a combination of wind and snow loading. Since the net result of such a combination of loading required a downward load on the roof, the 16 hydraulic rams acting on the roof surfaces were reversed. During this interlude, the plaster telltales were rephotographed to record the extent of cracking which had taken place under the applied wind loads.

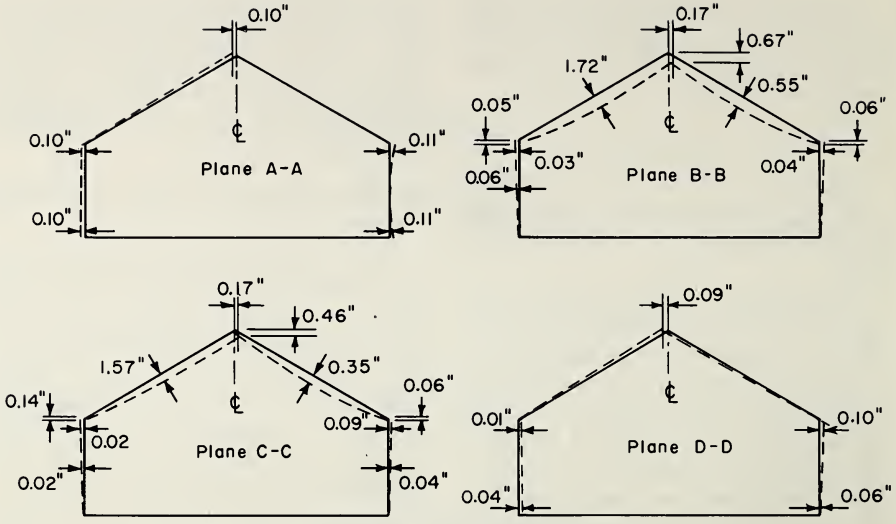
One-half Design Snow Load plus Wind Loads:

The combined loading of $\frac{1}{2}$ design snow plus wind load was applied for wind velocities varying from 70 to 120 mph. At 70 mph sustained loading, a small new crack appeared on the ceiling side of telltale No. 4. No further cracking took place in the plaster telltales as regular increases of 10-mph wind velocity were applied until an equivalent condition of a 110-mph wind velocity was reached, when a small new crack became visible in telltale No. 8. A final loading of $\frac{1}{2}$ design snow loading plus a wind velocity of 120 mph was reached without any further apparent cracking in the telltales. After being maintained for a certain period, all loads were reduced to zero in three stages as before. The deformations resulting from this phase of loading for wind velocities of 90 and 120 mph are shown in Figs. 12 (*a*) and (*b*).

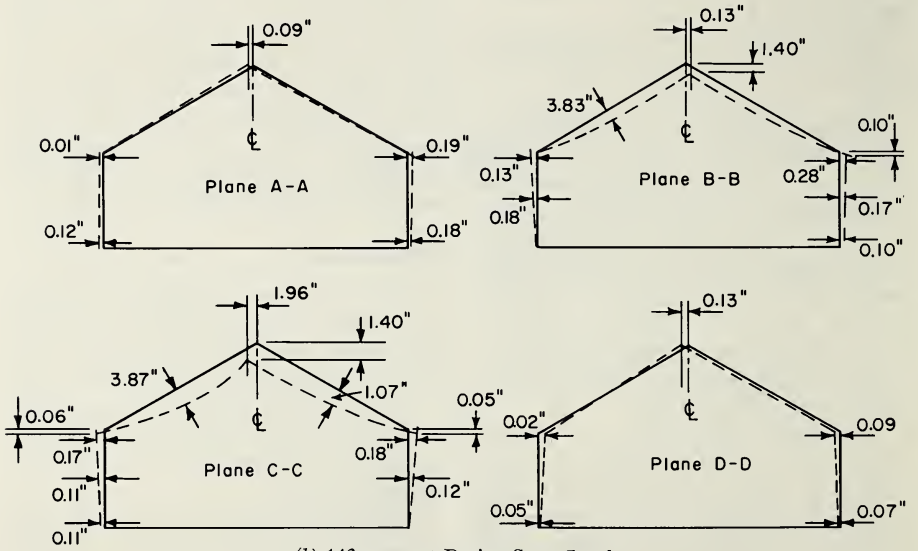
Snow Loads:

The next phase of loading consisted of the application of simulated snow loads which were applied in 25 per cent increments of the design snow load of 50.8 lb per sq ft of horizontally projected area for the Ottawa area as determined from the 1953 edition of the National Building Code.

As a first increment, the console pressures were raised in the two pressure channels actuating the hydraulic rams on the roof to a load equivalent to a 50 per cent design snow load. After a period of sustained loading, the crack which had appeared in telltale No. 9 earlier in the



(*a*) 100 per cent Design Snow Load.



(*b*) 143 per cent Design Snow Load.

———— Undistorted Frame
— — — Distorted Frame

FIG. 13.—Deformations due to Snow Loads.

NOTES.—Refer to Figs. 5 and 7 for loading and deflection points. Deformations are not drawn to scale.

Wind Direction: From the right.

test extended noticeably. The deformations were large in comparison to those of previous loadings. As the loading was increased from 50 to 75 per cent of the design snow load many new cracks occurred in the telltales, and sounds of cracking could be heard in the house. At telltale No. 9, the exterior wall separated from the solid plaster partition to the extent of $\frac{1}{8}$ in. near the ceiling. After $\frac{1}{2}$ hr of sustained loading, a further increase was made to 100 per cent design snow load. This loading was sustained for approximately 1 hr without any further serious damage. During the next increase from 100 to 125 per cent design snow load, further sounds of distress were heard, and the increase in the deformations shown on the deflection chart indicated that a failure was imminent. After $\frac{1}{2}$ hr had elapsed, the snow loading was again increased, but before 150 per cent snow loading was reached the loading had to be stopped because of the large deflections occurring (Figs. 13 (*a*) and (*b*)).

An inspection of the interior of the attic showed that all the collar ties were buckling horizontally to the extent of approximately 12 in. Under the maximum load reached (143 per cent of the design snow load), two collar ties snapped suddenly, one at the end of the house and one over the living room and kitchen. A splice in the longitudinal member attached to the underside of the collar ties had also given way. The break in the splice of the longitudinal member and the buckling of the collar ties is shown in Fig. 14 and the failure of the two end collar ties in Fig. 15. An additional failure occurred in one of the rafters at a knot. Except for a brief period of time during which failures were taking place in the roof assembly, it was possible to maintain stable console pressure for approximately 1 hr at 143 per cent design snow loading. During this period of sustained loading, careful observations were made of the interior of the test house and photographs were taken of the damage to the interior finishes and to the roof members. When this information had been gathered, all loads were gradually released and this phase of loading was completed.

FIG. 14.—Failure in Splice of 1- by 4-in. Longitudinal Member and Buckling of Collar Ties in Attic under 143 per cent of Design Snow Load.

FIG. 15.—Failure of Two Collar Ties in the Attic at one End of Test House under 143 per cent of Design Snow Load.

DISCUSSION OF RESULTS

Wind Loads—Internal Suction:

No apparent damage resulted from wind loads (internal suction) equivalent to wind velocities up to 90 mph. The first crack in a telltale occurred when this loading to 90 mph was repeated. Further

cracks became visible when the loads were increased up to 120 mph. This cracking was more pronounced at the junctions of the exterior wall and the solid plaster partitions than at other partitions.

The lateral movement under this wind loading was small (maximum 0.12 in.). The largest measured deflection in the rafters was 0.31 in. (outward at midspan).

Wind Loads—Internal Pressure:

During the application of the more severe conditions of wind loads with internal pressure some of the existing cracks in the telltales were extended slightly.

The greatest deflection in the rafters for a wind velocity of 120 mph was 0.80 in. (outward at the midspan). The greatest lateral movement in the exterior walls was 0.08 in.

One-half Design Snow Load plus Wind Loads:

Under the combined loading of wind and snow there was no further significant damage caused to the test house. This condition of loading produced a reversal of the direction of loading on the roof because of the downward snow loading. As the wind loading was increased, the rafter deflections decreased because of the uplift of the wind on the roof. Consequently the largest deflections in the roof under the combined loading were inward and occurred at the lowest wind velocity simulated in the test.

Snow Loads:

As the snow loads were applied in the final phase of the loading test, the deformations in the test house became more and more pronounced until failure began to occur in some parts of the roof at 143 per cent of the design snow load of approximately 50 lb per sq ft so that the load could not be increased any further.

At 100 per cent snow load the largest deflection in the rafters reached 1.72 in. on the front slope and 0.55 in. on the rear slope. The allowable deflection (National Building Code of Canada, 1953) for a rafter having a horizontally projected span of 12 ft 2 in., is $\frac{1}{240}$ of the horizontal projection of the rafter or 0.61 in., neglecting the support offered by the collar ties. Therefore, in the test, the allowable deflection was exceeded on the front slopes by 182 per cent at design snow loading for this particular roof slope. When failures occurred in the end collar ties and one rafter at 143 per cent of the design snow load, the rafter deflections had reached values of 3.87 in. on the front slope and 1.07 in. on the rear slope in plane C-C and there was a total of 1.96 in. of displacement in the ridge board towards the front slope on this plane of deflections.

The longitudinal 1 by 4-in. member which was fastened to the underside of the collar ties was not attached to the gable ends nor, because of the central location of the chimney, was it continuous over the full length of the house. Consequently as the snow load was increased, the collar ties were free to buckle in groups in each end of the house on opposite sides of the chimney, thus affording little support to the rafters after initial buckling had taken place. Figure 14 shows the failure of the splice in the longitudinal 1 by 4-in. runner and the extent of lateral buckling.

In the interior of the house, damage became progressively larger as the snow load was increased, particularly at the intersection of interior partitions and exterior walls, where the exterior walls moved outward at eave level in most cases. This pattern of deformation was produced largely by the spreading effect of the deflecting rafters and by ineffec-

FIG. 16.—Plaster Telltale No. 4 After Completion of Test, Showing Extent of Cracking but no Signs of Separation Between Adjacent Walls.

tive resistance of the ceiling joist to this movement. Already at 75 per cent of design snow loading the exterior wall and the solid plaster partition separated for a considerable distance from the ceiling down. Under further loading this separation grew larger. Although there was extensive cracking in the plaster telltales at other points in the test house, there were no signs of apparent separation at the junctions of the conventional partition walls and exterior walls, as shown in Fig. 16. Further evidence of the spreading of the exterior walls is shown in Fig. 17 where the front wall moved outward between the conventional partition walls of the entrance and produced a fracture in the gypsum board ceiling.

Alterations to Roof to Increase Resistance to Snow Loads:

As the relatively early failure of the roof at 143 per cent of the design snow load (or 72 lb per sq ft) seemed to be mainly the result of the compression in the collar ties, it was decided to alter this part of the roof frame to find if in a simple manner the strength of the roof could be improved. As a first possibility in preventing the buckling of the collar

FIG. 17.—Damage Caused to Ceiling Above Entrance by Spreading of Rafters Under Snow Load.

ties the runner board of the collar ties was cross-braced in each half of the house on both sides of the chimney. The cross-bracing consisted of 1 by 6-in. members nailed to the underside of the collar ties. Then the simulated snow loading of the roof was repeated and a maximum load of 175 per cent of the design snow load was reached, an increase of 32 per cent. At this point, a failure occurred in two rafters and one collar tie, and the loads on the roof had to be released.

After reviewing the considerable gain in strength obtained from the bracing of the existing collar ties it was decided to go a step further and replace the relatively thin 1 by 4-in. collar ties by 2 by 4-in. members at every second rafter, thus using no more lumber than before. These new collar struts were also cross-braced by a 1 by 4-in. runner and 1 by 6-in. cross-braces. The snow loading on the roof was then repeated again and a load of 206 per cent of the design snow load reached. At this point several rafters broke, two of which had been spliced after the earlier failure. Under this load, considerable further cracking inside the house was observed and photographs of all plaster telltales were taken. The vertical crack between the solid plaster partition and the outside wall was approximately $\frac{1}{2}$ in. wide, indicating the outward movement of the walls under the thrust developed by the rafters.

CONCLUSION

The results of the structural tests conducted by the Division of Building Research may be summarized as follows:

Resistance to Wind Loads (Racking Strength):

The application of simulated wind loads (wind direction towards front of house only) corresponding to velocities up to 90 mph (design speed) and 120 mph (approximately 80 per cent overload) proved that the house was well capable of withstanding these loads.

The movements of the end walls (parallel to the wind) due to wind loads were small. This was shown both by the deflection measurements and by the fact that of the four plaster telltales in the corners of the house only one cracked (No. 5). It was thus shown that the let-in 1 by 4-in. corner bracing, combined with the various finish materials, provided sufficient racking strength and that exterior wall sheathing was not required for additional racking resistance (if 1.8 times the design wind load is considered an acceptable test load).

Resistance to Snow Loads:

The application of simulated snow loads (symmetrical loading only, design load approximately 50 lb per sq ft) showed that the house in its original form could only carry 143 per cent of the design snow load and that, under 100 per cent design load, excessive deflections occurred in the roof structure. This indicated the following:

(1) *Rafters.*—The 2 by 4-in. rafters on 16-in. centers were structurally inadequate. It should be noted that these rafters do not meet the requirements of the National Building Code of Canada (1953) nor those of the present Building Standards of the Central Mortgage and Housing Corporation of Canada, although, for the latter, the 1 by 5-in. collar ties might be interpreted as intermediate supports and thus make the 2 by 4-in. rafters acceptable to those standards.

(2) *Collar Ties.*—The 1 by 5-in. collar ties were subjected to compressive loads (although their name implies a tension member). They were not able to resist these compressive loads effectively due to early buckling. The 1 by 4-in. longi-

tudinal runner attached to the underside of the collar ties was of little value in preventing them from buckling, because it was not continuous nor adequately braced against longitudinal movement.

After failure of the roof in its original form under 143 per cent of the design snow load the collar ties were altered to make them "collar struts" without using more lumber by replacing the 1 by 5-in. members fastened to every rafter pair by 2 by 4-in. members at every second rafter pair and cross-bracing. This increased the load-carrying capacity of the roof by 63 per cent to 206 per cent of the design snow load. It appears that in this form the collar struts can justifiably be considered as intermediate supports for the rafters.

(3) *Exterior Walls.*—Under snow loading separations began to occur at the junction of the rear exterior wall and the solid plaster partition walls under 75 per cent of design snow loading, indicating that the thrust of the rafters tended to deflect the exterior side walls outward at the ceiling level at points where insufficient resistance was afforded by the ceiling joists and the interior cross partitions. The connections between the rear exterior wall and the interior solid plaster partitions were structurally inadequate in preventing this separation.

Further Tests:

The evaluation of the strength and rigidity of house structures is difficult because of their complexity of form, variations in materials, workmanship, method of construction, and many other factors which make direct comparisons questionable. Before definite conclusions can be reached, more information is needed and a number of structural tests are necessary. The investigation carried out on this experimental house has been the first step by the Division in obtaining this information.

Future tests may be carried out in the laboratory on special full-scale test houses rather than in the field, because of the amount of time and cost involved in building up the test equipment around an existing house as was done in this first test.

Acknowledgments:

The authors wish to express their thanks to all those whose time and effort contributed to the successful completion of this project. To Mr. R. F. Legget, Director of the Division of Building Research, with whose approval this paper is published, acknowledgement is gratefully made for his encouragement and assistance in this project.

REFERENCES

(1) Central Mortgage and Housing Corporation, *Building Standards* (Excluding Apartment Buildings), Ottawa, Canada. Reprinted April 1954.

(2) Housing and Home Finance Agency, *Performance Standards* (Structural and Insulation Requirements for Houses), Housing and Home Finance Agency Technical Office, Washington 25, D. C. June 1947.

(3) David J. Isaacs, "The Structural Sufficiency of Domestic Buildings," *Bulletin* Department of Works and Housing, Commonwealth Experimental Building Station, North Ryde, Sydney, N. S. W., No. 1, June 1946.

(4) J. A. P. Laurie and Charles A. Rigby, "Some further Investigations into the Structural Requirements of Single Storey Urban Bantu Houses," *Bulletin* South African Council for Scientific and Industrial Research, Pretoria, South Africa, No. 12, June 1954.

(5) Ministry of Works, "House Construction," *Post-War Building Studies No. 1*, His Majesty's Stationery Office, London (1944).

(6) Ministry of Works. "House Construction,"

Post-War Building Studies No. 23, His Majesty's Stationery Office, London (1946).
(7) N. D. Nathan, "Structural Requirements of Single-Storey Native Houses," *Bulletin* South African Council for Scientific and Industrial Research, South Africa, No. 9, Dec. 1952.
(8) "National Building Code of Canada 1953," National Research Council, Canada, Ottawa (1953).
(9) A. Short and L. G. Simms, "Survey of Loading Tests on Some Post-War Prototype Houses," *The Structural Engineer*, Feb. 1949, pp. 67–98.
(10) F. G. Thomas, "Structural Engineering Research at the Building Research Station," *The Structural Engineer*, Vol. XXVI, No. 2 (1948).
(11) F. G. Thomas, "Structural Requirements for Houses," *Special Report, No. 1* Department of Scientific and Industrial Research, National Building Studies (1948).
(12) U. S. Naval Civil Engineering Research and Evaluation Laboratory, "Evaluation of a 40-ft by 100-ft Arch Rib Prefabricated Metal Utility Building," *Final Technical Memorandum M-039*, Port Hueneme, Calif. May 1, 1952.
(13) U. S. Naval Civil Engineering Research and Evaluation Laboratory, "Evaluation of a 40-ft by 100-ft Frameless Straight Sided Prefabricated Metal Utility Building," *Final Technical Memorandum M-050*, Port Hueneme, Calif., April 1, 1952.
(14) U. S. Naval Civil Engineering Research and Evaluation Laboratory, "Evaluation of a 20-ft by 48-ft Arch Rib Prefabricated Metal Barracks Building," *Final Technical Memorandum M-055*, Port Hueneme, Calif., April 1, 1952.
(15) H. L. Whittemore and Ambrose H. Stang, "Methods of Determining the Structural Properties of Low Cost House Construction," National Bureau of Standards, Building Materials and Structures, Report No. 2, Aug. 10, 1938.
(16) C. S. Yeates, "The Application of Testing Methods to Modern Building Techniques," *Engineering Inspection*, Vol. 12, No. 4, pp. 4–25, (1948).

DISCUSSION

MR. T. K. MAY.[1]—The framing members, of course, are the principal concern. Would the authors describe the framing members as being of high-grade, average-grade, low-grade or mixed grades of lumber?

MR. W. R. SCHRIEVER (*author*).—They were of average grade.

As I mentioned in the beginning, no special attempt was made to differ in any way in this house from the normal construction methods used in our area.

CHAIRMAN ROBERT F. LEGGET.[2]—May I ask what species?

MR. SCHRIEVER.—Pine.

MR. R. F. BARTELMES.[3]—The authors mentioned 120 mph resistance capability for the building, that is on the framing. What was the method of cleating the siding on to resist suction pulls?

MR. SCHRIEVER.—We did not investigate the resistance of the cleating to that suction. Originally we were trying to include a study of this factor by attaching the roof bearing pads on the aluminum by gluing. However, during the test we found that the glue which we had used was not able to resist the pull of the bearing pads and, to end our troubles of repairing the pads all the time, we finally drilled through the roof and used wire loops which went around the bearing pads and rafters.

MR. L. J. MARKWARDT.[4]—Was the effect studied on deformation of reversing the stresses by changing the direction of the loading?

MR. SCHRIEVER.—No. We applied it

[1] Director of Technical Service, West Coast Lumbermen's Assn., Portland, Ore.

[2] Director of Building Research, National Research Council of Canada, Division of Building Research, Ottawa, Ont., Canada.

[3] Chief, Building Design Branch, Civil Engineering Dept., U. S. Corps of Engineers, Research and Development Lab., Fort Belvoir, Va.

[4] Asst. Director, Forest Products Lab., Forest Service, U. S. Dept. of Agriculture, Madison, Wis.

only from one direction. The only reversal we did was from wind load to snow load.

MR. R. F. LUXFORD.[5]—Was the interior wall covering wet plaster construction or was it dry wall construction?

MR. SCHRIEVER.—It was dry wall construction.

MR. LUXFORD.—Did you use tape over the joints?

MR. SCHRIEVER.—Yes, I believe all joints were taped.

MR. IRA ASHFIELD.[6]—In reply to Mr. Luxford's question, gypsum wall board with standard cement joints and paper tape was used with the one exception that there is the plaster partition.

MR. LUXFORD.—I was speaking of exterior walls.

MR. ASHFIELD.—They were gypsum board on wood studs.

MR. JOHN F. LEWIS.[7]—I was concerned with the roof bracing system shown in Figs. 2 and 14. The use of collar ties is not particularly conventional to Southern California or to building codes and housing agencies in the States, and I wondered if that was a typical system used in Canada, or if it was tested in that way as an experimental basis of determining what might happen with just the collar ties. Conventional framing to me is made of struts and kickers and complicated bracing system that we see in most housing projects.

MR. ASHFIELD.—I do not accept the blame for the design, but I can report about it.

We selected this particular system because it was the lightest of three or four methods which were in reasonably common use, and we wanted to find out how it would perform. I think the authors have reported on that.

CHAIRMAN LEGGET.—As a result of this work, the Division of Building Research has embarked on a very extensive program of study of standard Canadian roof frame designs for house structures, as one of the several by-products of this work.

MR. J. E. DYKINS.[8]—I would like to ask Mr. Ashfield what nail spacing was used on the wallboard?

MR. ASHFIELD.—We followed the manufacturers' directions, and I believe the nails were 6 to 9 in. apart.

---

[5] Head, Housing Section, Forest Products Lab., Forest Service, U. S. Dept. of Agriculture, Madison, Wis.

[6] Supervisor, Building Materials Dept., Central Mortgage and Housing Corp., Ottawa, Ont., Canada.

[7] Structural Research Engr., Dept. of County Engineer, Building and Safety Div., County of Los Angeles, Los Angeles, Calif.

[8] Project Engineer, Environmental Division, U. S. Naval Research Establishment, Port Hueneme, Calif.

FULL-SCALE TESTS OF PRE-CAST MULTI-STORY FLAT CONSTRUCTION

By A. J. Francis,[1] W. P. Brown,[2] and S. Aroni[3]

Synopsis

Full-scale tests were carried out as part of an investigation into the possibility of extending a precast, two-story flat construction to three and more stories. The load-bearing walls, lightly bolted together, had no positive connection to the floor slabs, and the main object of the tests was to observe the behavior of the structure under lateral loading.

The three-story unit tested, with additional dead load to simulate a further $1\frac{1}{2}$ stories, was subjected to a horizontal uniformly distributed load, applied at the top of the third story walls. This was designed to represent a wind load of 75 mph on a five-story flat; the maximum horizontal load applied being equivalent to a 90 mph wind.

Appropriate measurements were taken to record the lateral deflection of the building and the movement of the walls relative to the floor slabs. These measurements, together with the observed cracks, indicated a distortion of the unit, with tilting and sliding of the walls.

As a result of the information obtained from the tests, positive connection between walls and floor slabs, for multi-story construction, was recommended.

The need for further research is briefly discussed.

The Housing Commission of Victoria, Australia, has for some years employed a precast type of construction for its concrete houses. Several designs are in use, including the two-story block of four flats shown in Fig. 1. In this type, each story consists of 4 in. thick outer and spine wall units, which are load-bearing, and 3-in. non-load-bearing partition wall units, the first floor being of $4\frac{1}{2}$-in. thick slab units with a mortar key between each, spanning between either front or back wall and spine wall. The walls are provided with light, centrally-placed, single-layer reinforcements consisting of plain bars between $\frac{3}{8}$ and $\frac{3}{4}$ in. diameter.

The construction is virtually "dry". The wall units are lightly connected where they butt together by three $\frac{1}{2}$-in. bolts, but no other positive key or connection is provided between wall units or between floor slabs and walls. Horizontal and vertical joints are sealed with mastic and pointed. Sheet lead, 3 lb per sq ft, is used to give even bearing at top and bottom edges of all wall units where they are in contact with floor slabs. Ground floor walls are normally sup-

[1] Professor of Civil Engineering, University of Melbourne, Melbourne, Australia.

[2] Consulting Engineer, Melbourne, Australia.

[3] Assistant Lecturer, University of Melbourne, Melbourne, Australia.

ported at either end of each unit by pier foundations, but in very bad ground a continuous footing may be provided.

All wall and floor units are factory-produced at the Holmsglen Housing Factory, Melbourne, where production runs at the rate of 25 houses per week.

In 1955, the first two authors were asked by the commission to examine the possibility of extending this form of construction to three or more stories.

PRELIMINARY STUDIES

Analyses were made of the structural performance of the design shown in Fig. 1 when extended beyond two stories in height, the design live load on floors being 40 lb per sq ft and the lateral wind pressure on front and back walls 11.3 lb per sq ft (windward) and 5.6 lb per sq ft (leeward), as laid down in the Uniform Building Regulations of Victoria (U.B.R.).

Briefly, it was found that with continuous footings the design could be extended, without increased wall thickness or reinforcement, to at least 10 stories, at which stage the compressive stress in the walls under wind plus floor live load could become excessive under the worst

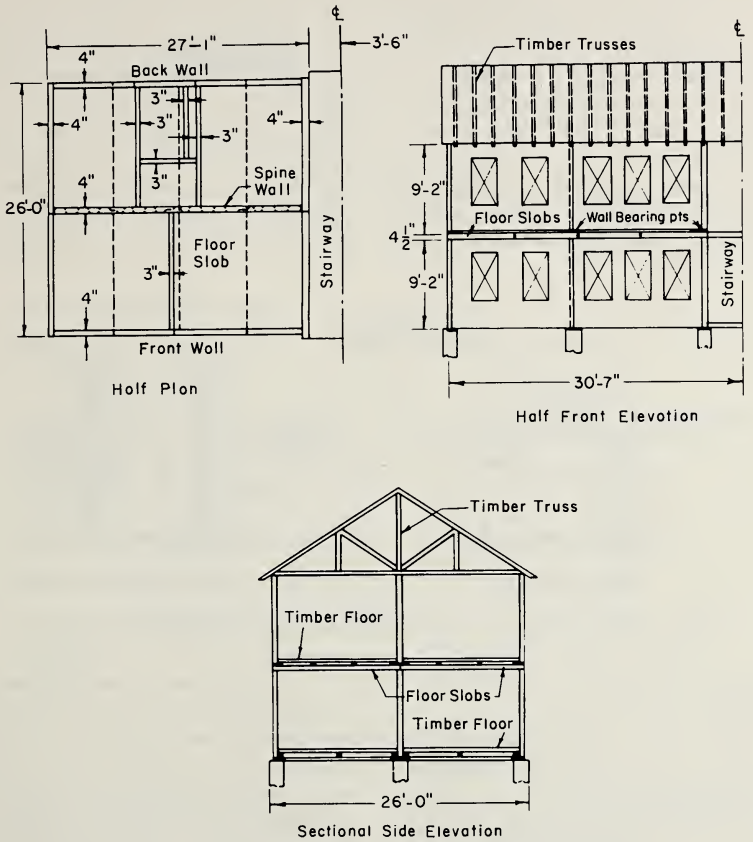


FIG. 1.—Details of Two-Story Four-Flat Unit.

conditions of wall support which it was reasonable to assume. Lateral stability would not be endangered below a height of sixty-eight stories. As is normal in Victoria, earthquake effects were not considered.

It was realized that the problem involves a number of unknown factors which cannot be taken account of in calculation, such as: (1) the actual performance and compressive strength under eccentric load of wall panels, under sustained as well as static loading; and (2) the extent to which a multi-story structure, while being amply strong, might develop undesirable features such as opening out of joints, particularly in walls, under load.

Proposals for research relating to the first factor are now under consideration. The commission accepted a recommendation, concerning factor two that full-scale tests, involving lateral wind loading, to as many stories as practicable should be carried out on a single flat unit. The tests, which were made in February 1956 on a site near Holmsglen, are described below.

OUTLINE OF TESTS

Unit Tested:

The main object of the tests was to determine the behavior under wind plus dead load of as high a unit as possible, and a five-story unit was decided upon. Floor live load was omitted since the lateral behavior is more critical without it. A three-story unit, 27 ft by 26 ft in plan, this being the highest that could be erected with the available cranes, was built. Normal construction procedure was adopted, and isolated footings as shown in Fig. 1 were employed. The end walls of the third story were of first-story type (not gabled). Five complete layers of floor slabs were then laid on the uppermost story walls, the layers being separated by timber battens at the supports over the outer and spine walls.

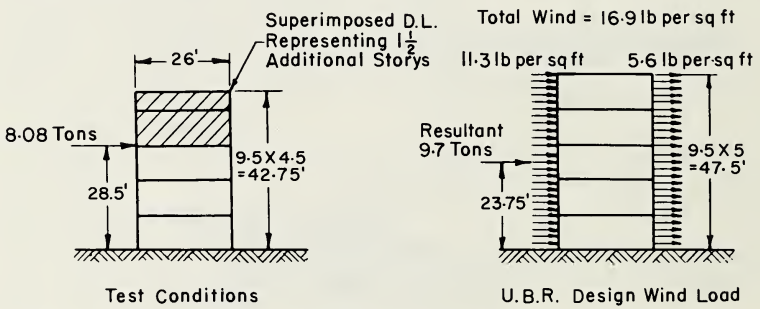


FIG. 2.—Test Conditions, Uniform Building Regulations Design Wind Load.

The superimposed load of these slabs was equivalent to the dead load of a further 1½ stories acting directly on the load-bearing outer and spine walls. This limitation on the superimposed load was set by high bearing stresses at the base of the ground story walls near the supports.

Undisturbed samples were taken from the foundation at depths down to 11 ft 6 in. below pier bases, and it was concluded that there was an adequate margin of safety against both shear and settlement failure.

Test Procedure:

Method of Loading.—It was desired, as already mentioned, to simulate the lat-

eral load caused by wind acting on a unit five stories high. For obvious reasons, however, it was not feasible to apply a uniformly distributed load over the whole story unit. A full comparison of the moments and shears actually produced in the tests by a lateral load of 18, 100 lb (called the design wind load) and those

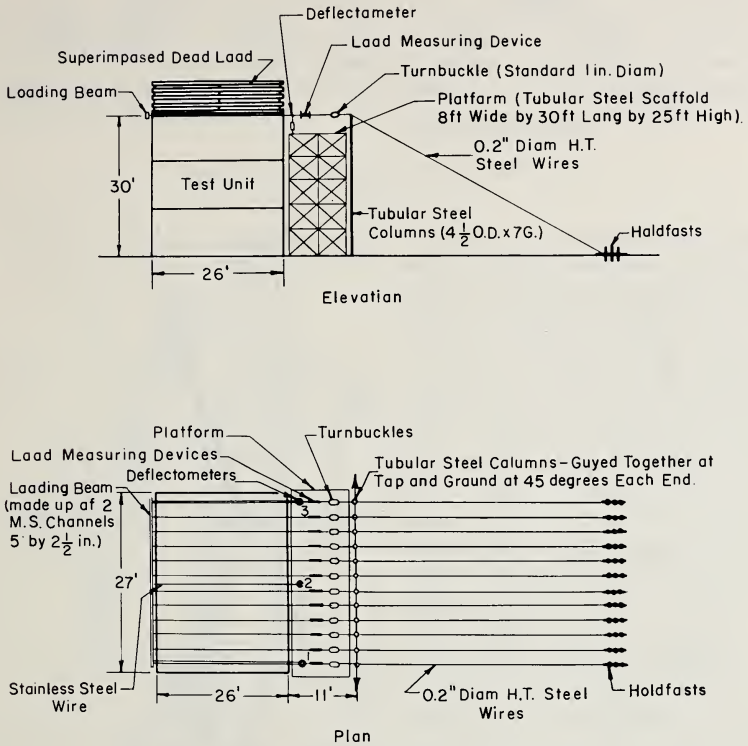


FIG. 3.—Details of Method of Lateral Loading.

area of the front and back walls, and instead a horizontal load of 18, 100 lb, uniformly distributed along the 27-ft length of the building, was applied at the top of the third story walls (Fig. 2). This gave the correct design overturning moment according to the provisions of the U.B.R. at the base of the ground story, but rather less than the correct horizontal shear at the base. On the upper floors, on the other hand, the horizontal shear was in excess of that produced by the U.B.R. design wind load at these levels in a five-

due to the U.B.R. design load is shown in the following table:

Level	Design Wind Load in Tests		U.B.R. Design Load	
	Moment, ft lb	Shear, lb	Moment, ft lb	Shear, lb
Third floor. . . .	171 500	18 100	186 000	13 000
Second floor. . .	344 000	18 100	330 000	17 300
Ground floor. . .	517 000	18 100	517 000	21 650

The maximum lateral load applied during the tests was 1.5 times the design wind load.

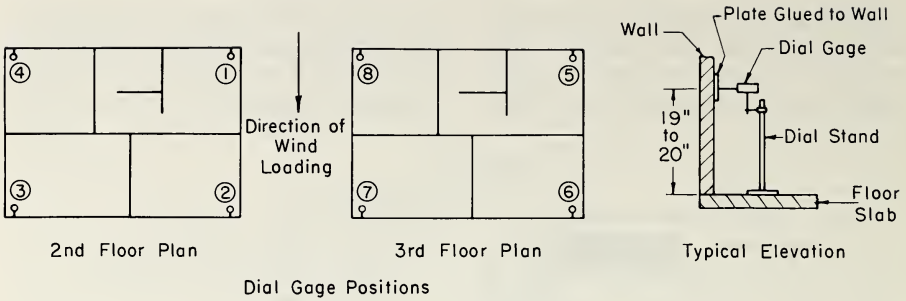


FIG. 4.—Measurements Recorded in Tests.

Figure 3 gives details of the method of lateral loading. Load was applied to the windward side of the building through twelve 0.2-in. diameter high-tensile steel wires equally spaced along the length of the building. The wires were stayed back to the ground by Army type holdfasts (anchors), the vertical reactions at the change in direction being carried by the tubular columns shown in Fig. 3. The load in each wire was applied by turnbuckle and measured by a simple load measuring device consisting of a short length of $\frac{3}{8}$ in. diameter high-strength silver steel rod with gage points 8 in. apart, to which a Demec strain gage[4, 5] could be applied. Each load meter was previously calibrated with the Demec gage in an Amsler testing machine. The error in reading the load in each wire by this means is unlikely to have exceeded ± 5 per cent, and the total load on the 12 wires was probably much less in error than this. A temperature correction was found to be necessary and was applied.

In applying an increment of load, the wires were tightened systematically, working from the center wire towards either end, so that the resultant load was as nearly as possible central throughout the operation. When all wires had been loaded, each load meter was rechecked and the wire load adjusted if necessary.

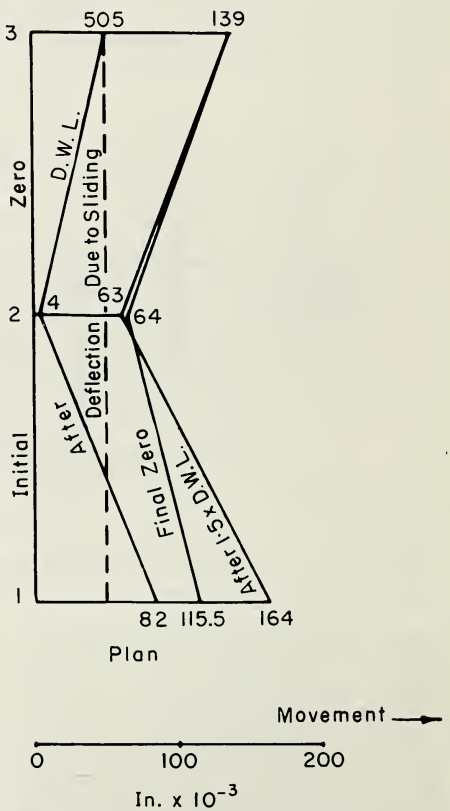


FIG. 5.—Lateral Deflection of Top of Building. Deflectometer Readings. For Position See Fig. 3.

[4] P. B. Morice, and G. D. Base, "The Design and Use of a Demountable Mechanical Strain Gauge for Concrete Structures," *Magazine of Concrete Research*, No. 13, Aug. 1953, pp. 37–42.

[5] G. D. Base, "Further Notes on the Demec, A Demountable Mechanical Strain Gauge for Concrete Structures," *Magazine of Concrete Research*. No. 19. March 1955, pp. 35–38.

Loading Procedure.—Tests were carried out on two days, Wednesday, February 8, and Friday, February 10. On the Wednesday, the design wind load was applied in two equal increments, various measurements as detailed below being taken at each increment, and the load was then increased by a further 50 per cent. This load was maintained for $3\frac{1}{2}$ hr and then reduced to zero.

On the Friday the design load was applied in one increment, and the 50 per cent overload applied in a second increment.

Measurements Taken.—(1) Lateral deflection of building at top of third-story walls was measured at three points (either end and center of windward wall) by means of deflectometers similar to those used at the Building Research Station, U.K., fitted with 0.001-in. dial gages and indicated in Fig. 3. Stainless steel wire was attached at each point to the wall and passed through the width of the building to the deflectometer stand and over a pulley attached to each stand.

(2) Lateral movement between base of windward and leeward walls and floor slabs was measured at each corner of the building on the second and third floors by means of 0.001-in. dial gages fixed to stands set on the floor, the plunger of each gage bearing against a polished aluminum alloy plate glued to the wall concrete (see Fig. 4).

(3) Strain measurement on the ground floor walls was also taken. Thirty-seven 8-in. gage positions were established on these walls for strain readings with the Demec gage. The application and removal of the wind load caused such a small change in stress that no useful results were obtained. Strain readings were taken before and after the dismantling of all the building above the bottom story.

(4) Visual observations were made of

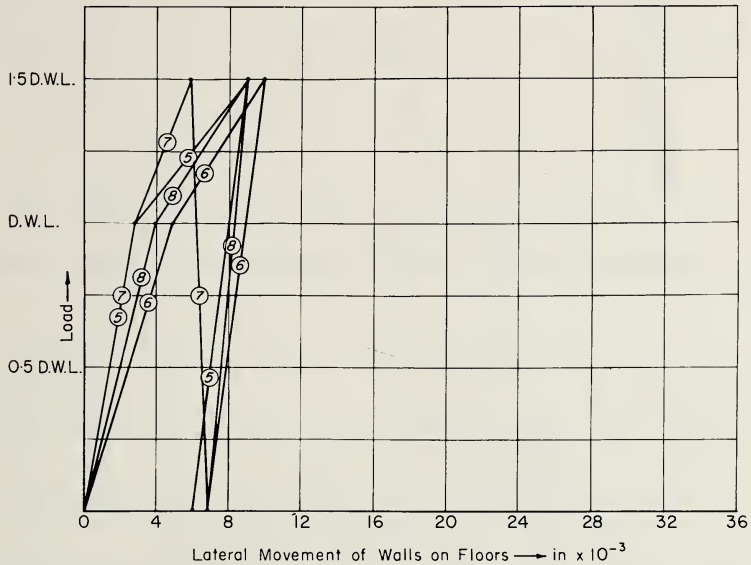


FIG. 6.—Load-Dial Gage Readings. Third Story. Friday Test. For Position of Dial Gages See Fig. 4.

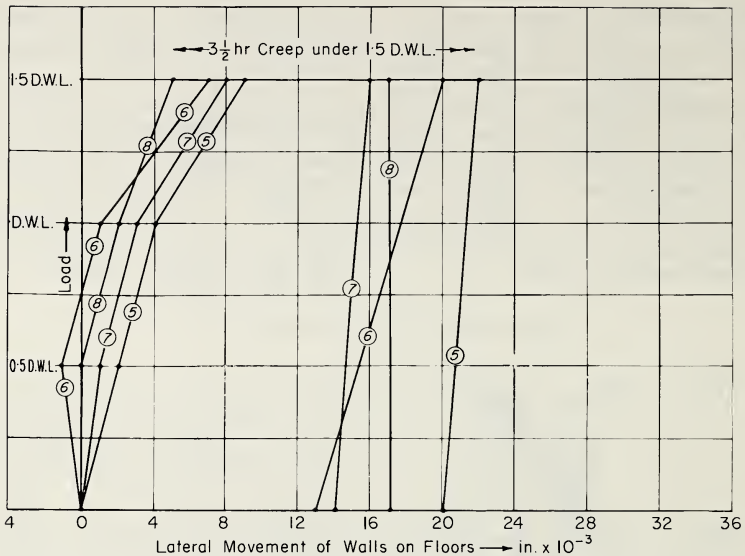


FIG. 7.—Load-Dial Gage Readings. Third Story. Wednesday Test. For Position of Dial Gages See Fig. 4.

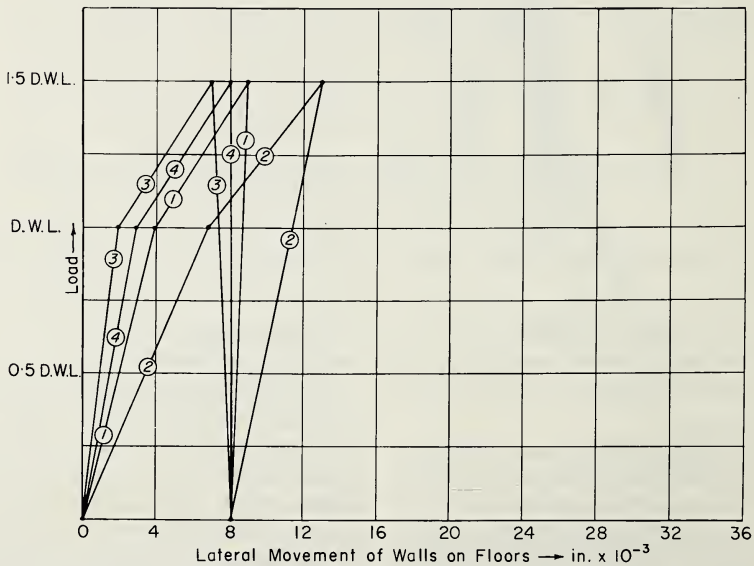


FIG. 8.—Load-Dial Gage Readings. Second Story. Friday Test. For Position of Dial Gages See Fig. 4.

movement on cracking between walls and wall and floor slab, and in wall panel at points of high stress.

TEST OBSERVATIONS AND DISCUSSIONS

Lateral Deflection of Top of Building:

The results obtained during the Friday test are shown in Fig. 5. It will be seen that the lateral deflection was very much greater than the deflection (0.06×10^{-3} in. at design wind load) that would take place if the structure behaved as an elastic monolithic box frame, taking account only of the outer walls in the calculation. A further point of interest is that the recovery on unloading was small; it was nil at point 3 and maximum (48.5×10^{-3} in.) at point 1. This indicates a distortion of the unit during load and subsequent recovery.

Lateral Movement of Walls on Floors:

Figures 6 to 9 give graphically some of the measurements recorded. The figures in circles indicate the points at which measurements were taken (Fig. 4). The average bearing pressure on the third floor at 50 per cent overload was 448 lb per sq in. Assuming that the lateral shearing force was uniformly distributed among the various bearing surfaces, the average static coefficient of friction required to prevent sliding at the third floor was 0.104. For concrete on concrete the value normally adopted is about 0.7, so that there is an ample margin of safety. Lead however, creeps appreciably at normal temperatures at stresses as low as 224 lb per sq in.[6] The temperature during both tests was high (>90 F) and the relatively large movements recorded are evidently due to creep in the lead. The lack of recovery on removal of load supports this view (see especially Fig. 8).

Sliding in this building can occur at six levels (between each floor and the walls above and below it). If the move-

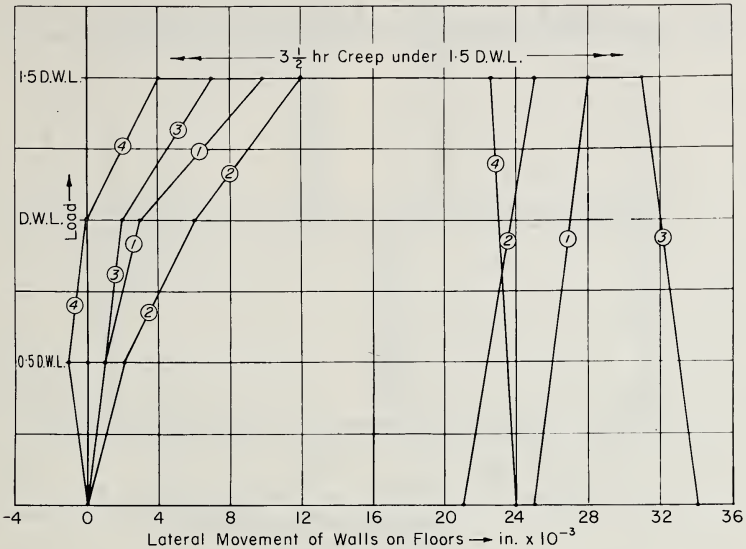


FIG. 9.—Load-Dial Gage Readings. Second Story. Wednesday Test. For position of Dial Gages See Fig. 4.

[6] Reiner, "Building Materials," p. 132 (1954).

ments actually recorded at two of those levels are typical of the others, the part of the lateral deflection of the building due to this sliding may be conjectured to be as shown by the dotted line in Fig. 5. It is evident that sliding on the lead pads was not the predominant cause of the lateral deflections observed; tilting of the rangment would also serve to align the walls in the different stories accurately over each other.

Strain Measurements on the Ground Floor Walls:

The results obtained were inconclusive owing to the strains caused by tempera-

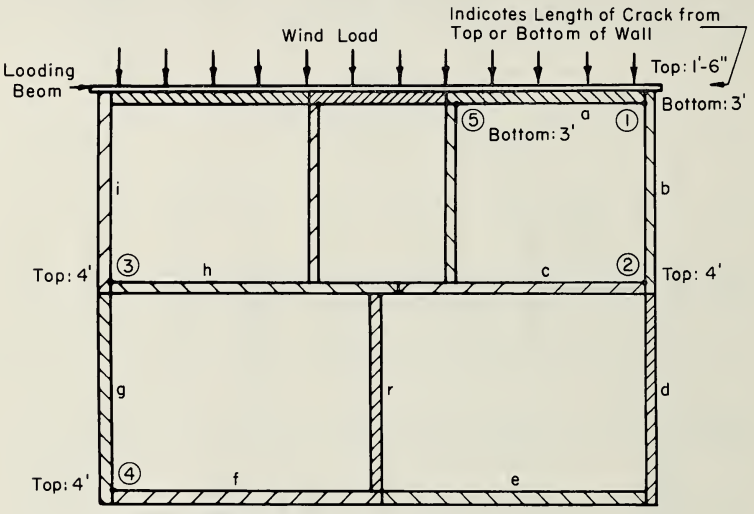


FIG. 10.—Details of the Extent of Cracking.

walls (which was not separately measured) must have accounted for much of the movement.

There would be no difficulty in eliminating the sliding altogether by means of suitable keys between walls and floors. For instance, two $\frac{1}{2}$-in. mild steel dowel pins per wall unit, fully grouted up, would be ample. This or some other arture changes and differential effects being of the same order as the dead load strains.

Signs of Movement Within the Building:

During loading, cracks appeared in the third story (but not elsewhere) in the mortar joints between end walls and front, spine, and back walls. Figure 10 gives details of the extent of cracking.

The estimated width of the cracks is also shown.

These movements are consistent with the sliding, tilting, and distortion discussed above. In each story there was a relative lateral movement of ceiling and floor owing to creep in the lead pads at top and bottom of the walls. Where the floor slabs rest on the end walls, direct sliding must occur, but the front, spine, and back walls will tilt rather than remain upright unless this tilting is prevented by the $\frac{1}{2}$-in. bolts between abutting wall units; and these are unlikely to restrain the tendency to tilting within the small range of movement recorded (see Fig. 11). The floors appear to act monolithically in each story.

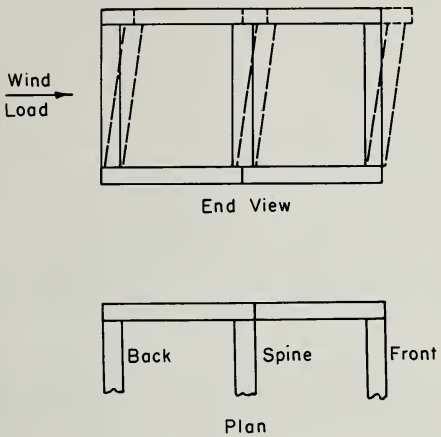


FIG. 11.—Type of Movement During Test.

Although some hair cracking of the concrete in the wall units was observed while the superimposed dead load was being added, no further opening of these cracks could be detected when the lateral load was applied; the increases in vertical stress in the front and back walls due to the lateral test load were, however, small (of the order of ± 200 psi).

A critical factor is obviously the concrete bearing pressure at the supports of the ground floor walls. Calculations showed an average bearing pressure of 550 psi with a maximum value of 1225 psi under one of the spinal wall supports. At a couple of these points of high bearing pressure, spalling was observed in the walls and also in the piers.

CONCLUSIONS

1. The lateral movements which developed in the structure were unexpected, and if maintained for a long enough time would have caused its collapse. At the same time it must be remembered that:

(*a*) At the upper floor levels the shearing force under design wind load was greater than would result from a normal (uniformly distributed) wind pressure, being the same at each level instead of diminishing with height.

(*b*) A wind of 75 mph is required to produce the U.B.R. design wind pressure. It is very unlikely that a wind of this velocity would be sustained for any appreciable period of time—such wind pressures in Melbourne usually occur in short-term gusts lasting a matter of seconds. There would therefore be little opportunity for the lead to creep. The high ambient temperatures during the tests (80 to 90 F) also influenced the amount of creep and consequent lateral movements that occurred, and these are not likely to be associated with very high winds.

(*c*) A 50 per cent overload was applied, which represents a wind velocity of over 90 mph. This speed has never been recorded in Melbourne.

It was recommended to the commission that, for complete safety, positive keys such as dowels should be provided between walls and floor slabs for construction greater than three stories. Since creep and the resulting wall tilt account for nearly the whole of the lateral movement, the lateral deflections with such keys in position should not appreciably exceed the elastic deflec-

tion; the cracking observed between wall units would also be prevented.

2. The self weight of the test unit and the superimposed weight of the added floor slabs are a little more than equivalent to the dead load and 1.5 times the floor live load of three stories. It may therefore be said that, as regards the walls and the foundation piers, the structure was proof tested in the course of erection. The structure in general behaved satisfactorily under this test, though there were indications of localized cracking in the ground story walls, especially at points of support and at internal angles at openings. The mean vertical compressive stresses in the walls under these conditions were of the order of 150 psi.

3. Overall stress or stability problems do not become serious in this type of construction until a large number of stories is attained, and in a test on a building with a limited number of stories extensive confirmation of theoretical predictions is not to be expected. Tests such as those described are, however, valuable in bringing out unexpected weaknesses (such as the excessive flow deformations of the lead pad bearings) or emphasize anticipated difficulties (such as the cracking at the base of the ground story walls at point of support). The tests also helped greatly in understanding how a building of this type could collapse laterally, and furnished supporting evidence for the belief that, provided precautions such as keying in of floors to walls are taken, monolithic construction at these points is not necessary for stability or to ensure monolithic action of the structure. Such action is ensured by its dead weight acting as a prestressing force.

4. Research is necessary on: (*a*) The strength and general behavior of wall units (plain or with openings) supported either at their ends or continuously, and subjected to compressive loads applied axially or eccentrically.

(*b*) The block stability of multi-story units of dry construction of this form.

(*c*) Foundation problems arising in load-bearing walls with continuous footings.

Acknowledgments:

The authors are grateful to the Housing Commission of Victoria for permission to describe the tests in this paper.

DISCUSSION

MR. E. CZERNIAK.[1]—Will Mr. Leggett please elaborate on the use of lead? I do not think it is rather conventional.

MR. R. F. LEGGETT[2] (*for the author*).—All I can say is that lead pads were used in order to effect adequate bearing, I presume because of the slight irregularities in the finish of the bottom of the slabs, to insure continuity and uniformity of bearing, lead being a material that would yield under the pressures involved.

MR. JAMES R. BLACKWELL.[3]—What was the weight per cubic foot of the concrete used in the slabs?

MR. LEGGETT.—It was hard rock concrete. I do not know the weight.

[1] Design Engineering Supervisor, Fluor Corp., Ltd., East Los Angeles Branch, Calif.

[2] Director of Building Research, National Research Council of Canada, Division of Building Research, Ottawa, Ont., Canada.

[3] Materials Engineer, Arizona Sand and Rocks, Co., Phoenix, Ariz.

11-21-58

Date Due

Due	Returned	Due	Returned
MAR 28 '67	MAR 16 '67		
OCT 31 '67			
MAY 22 '69	MAY 16 '69		
FEB 15 '72	FEB 15 '72		

Symposium on full-scale tests afa
620.112A512st no.210

3 1262 03006 1260

FINE ARTS
LIBRARY

one of many
g Materials in
knowledge of
ping standard
ch of the data
of many of the
from industry,

ed many tech-
oks. These may
reports by the
ations of data
many organiza-
blications and
rill be furnished